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Rensselaer Polytechnic Institute

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A STUDY OF MINIMUM BAR SPACING FOR
BOND IN THIN-SHELL PRECAST CONCRETE

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Annapolis, Md.





A STUDY OF MINIMUM BAR SPACING
FOR
BOND IN THIN-SHELL PRECAST CONCRETE

Submitted to

THE FACULTY OF RENSSELAER POLYTECHNIC INSTITUTE
in partial fulfillment of the requirements for
THE DEGREE OF MASTER OF CIVIL ENGINEERING

By

JOHN WALLACE COLLINS

GEORGE FREDERICK JUBB

HARRY HOWARD LOEFFLER, JR.

TROY, NEW YORK

JUNE, 1948

A STUDY OF MINIMUM BAY SPACING
FOR
ROOMS IN TWIN-SHELL PRECAST CONCRETE

Thesis
C6
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BY
JOSE SALVADOR COLLINS
GEORGE FREDERICK JUBB
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For his initial suggestion and outline of this subject and for his continued advice, the authors wish to express their sincere appreciation to Mr. A. Amirikian, Head Designing Engineer, Bureau of Yards and Docks, Navy Department, Washington, D. C.

In addition, appreciation is expressed to Admiral Louis B. Combs, Head of the Department of Civil Engineering, Rensselaer Polytechnic Institute, to Lt. Cdr. H. C. Kropf, Public Works Officer, Naval Supply Depot, Scotia, New York, and particularly to Mr. J. F. Throop of the Mechanics Department of Rensselaer Polytechnic Institute for his valuable suggestions and assistance.

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A STUDY OF MINIMUM BAR SPACING
FOR
BOND IN THIN-SHELL PRECAST CONCRETE

SYNOPSIS

The purpose of the tests was to determine the minimum bar spacing and clear cover required to develop bond in thin-shell, precast concrete.

The tests were of the pull-out type in which round bars of two sizes were cast in a horizontal position; clear spacing and cover (always equal) and the length of embedment were varied. The slips of the bars were measured at the loaded and free ends.

These tests indicate that for close spacing and thin cover the resistance to pull-out is a function of these variables and bar size for both deformed and plain bars, and in general, the greater the cover, the greater the bond resistance. Bars cast horizontally in thin sections with very stiff mixes are subject to water gain.

A STUDY OF ALUMINUM BAR SPACING

1933

BOND IN THIN-SHELL PORTLAND CONCRETE

SYNOPSIS

The purpose of the tests was to determine the aluminum bar spacing and clear cover required to develop bond in thin-shell, present concrete.

The tests were of the pull-out type in which round bars of two sizes were cast in a horizontal position; clear spacing and cover (always equal) and the length of embedment were varied. The slope of the bars were measured at the loaded and free ends.

These tests indicate that for clear spacing and thin cover the resistance to pull-out is a function of these variables and bar size for both deformed and plain bars, and in general, the greater the cover, the greater the bond resistance. Bars cast horizontally in thin sections with very stiff mixes are subject to water gain.

INTRODUCTION

The tests of pull-out specimens reported herein constitute the first phase of an investigation on bond efficiency in thin-shell precast concrete. The investigation was initiated on the suggestion of Mr. A. Amirikian, (24, 27)* Chief of the Design Section, Bureau of Yards and Docks, and was conducted by Lts. J. W. Collins, G. F. Jubb and H. H. Loeffler, all (CEC) USN, at Rensselaer Polytechnic Institute, Troy, New York.

Considerable investigation has been carried out in the study of bond since the turn of the century. Notable have been the works of Withey, Abrams, and Gilkey; and more recently, studies conducted by Watstein and Clark at the U. S. Bureau of Standards.

Bulletins by Withey (1, 2) in 1906 and 1909 were among the early contributions to the literature on bond. In 1913 Abrams (3) published his "Tests of Bond Between Concrete and Steel" which still stands as a classic on reinforced concrete research. In 1940 Gilkey's (18) "Bond Between Concrete and Steel" served to greatly stimulate research and thought toward a better understanding of the concept of bond. The numerous studies on bond have produced a variety of conclusions, some in agreement, some conflicting; but each has served to cast new light upon the variables involved. The subject is by no means closed.

* Numbers refer to articles as listed in Bibliography.

INTRODUCTION

The basis of pull-out specimens reported herein con-

sists of two phases of an investigation on bond

efficiency in thin-shell precast concrete. The investigation

was initiated on the suggestion of Mr. A. A. Artin, (24, 27)*

Chief of the Design Section, Bureau of Yards and Docks, and

was conducted by Mr. J. W. Collins, W. E. Davis and R. H.

Boettner, all (CSC) 001, at Research Polytechnic Institute,

Troy, New York.

Considerable investigation has been carried out in the

study of bond since the turn of the century. Research has

been the work of Whitney, Abrams, and Gilkey; and more recently,

studies conducted by Katsaris and Clark of the U. S. Bureau of

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in agreement, some conflicting; but each has served to cast

new light upon the variables involved. The subject is by no

means closed.

* Symbols refer to articles as listed in bibliography.

The advent of the use of thin-shell precast sections for use as girders, beams, and even in landing craft has begun, we believe, an era of utilization of prefabricated reinforced thin sections of concrete that will play an increasingly important part in construction of all kinds in the coming years.

The increasing use of thin sections has brought out the need for investigation of bond in thin sections, as few specifications cover the design of thin-shell sections with relatively large reinforcing bars.

Previous studies of bond utilizing the pull-out test have been single-bar specimens with greater than one-inch cover. The cover employed was in general not a fundamental variable, although Gilkey in his tests introduced this variable to a limited degree by placing some of the bars in an eccentric position. Gilkey and others have considered the relation between pull-out tests and beam tests, and more recently ACI Committee 208 (23) has reported tentative specifications for determining bond resistances using the beam method. This study utilized pull-out tests using three-bar specimens with cover ranging from 5/16-inch to 1-inch, and round deformed reinforcing bars of one type; two sizes, 3/4-inch and 1-inch.

Recognizing the variables in concrete, an attempt was made to hold constant all those except the specified variations in specimens. To this end all deformed bars were from the same lot and contained equal amounts of mill scale and rust. All cement was from the same lot; aggregate and sand were from the same vicinity, respectively. The casting of specimens was carried out with the exact repetition of procedure.

The advent of the use of thin-shell process sections for use as cylinders, beams, and even in landing craft has begun, we believe, an era of utilization of previously neglected materials. This sections of concrete that will play an increasingly important part in construction of all kinds in the coming years. The increasing use of thin sections has brought out the need for investigation of bond in thin sections, as the specifications cover the design of thin-shell sections with relatively large reinforcing bars.

Previous studies of bond utilizing the pull-out test have been single-bar specimens with greater than one-inch cover. The cover employed was in general not a standardized variable, although Gilkey in his tests indicated this variable as a limited degree by placing some of the bars in an eccentric position. Gilkey and others have considered the relation between pull-out tests and beam tests, and more recently ACI Committee 308 (23) has reported tentative specifications for determining bond resistance using the beam method. This study utilized pull-out tests using three-bar specimens with cover ranging from 5/16-inch to 1-inch, and round helical reinforcing bars of one type; two sizes, 3/4-inch and 1-inch. Determining the variables in concrete, an attempt was made to hold constant all those except the specified variations in specimens. To this end all reinforced bars were from the same lot and contained equal amounts of mill scale and rust. All cement was from the same lot; aggregate and sand were from the same locality, respectively. The casting of specimens was carried out with the exact repetition of procedure.

It is to be noted that preliminary results indicated a tension failure in the concrete due to the wedging action of the lugs on the deformed bars before a reasonable utilization of bond. This led to the casting of a batch of plain-bar specimens in an attempt to realize a higher bond efficiency as it was presumed that less wedging action would be present to cause splitting.

Considerable time-consuming preparation was involved in devising the procedure for casting the specimens and in the fabrication of the testing rig, the number of physical variables involved being considerable.

It is recognized that the use of a spherical bearing block, slotted to take the three bars in the testing yoke, would have eliminated any possibility of eccentricity in loading, but in lieu of availability, the method arrived at gave reasonably consistent results. It is regrettable that time did not permit the investigation of the two and one-bar specimens, however, the testing procedure and apparatus were designed to cover the complete series and it is hoped that this may be done in the future.

It is to be noted that preliminary testing revealed a reaction failure in the composite due to the varying degrees of cure on the different parts before a reasonable utilization of force. This led to the casting of a series of plates--specimens in an attempt to produce a higher bond efficiency as it was presumed that less testing action would be present to cause splitting.

Considerable time-consuming investigation was involved in devising the procedure for casting the specimens and in the fabrication of the testing rig. The number of physical attributes involved being considerable.

It is recognized that the use of a spherical bearing block, aligned to take the three bars in the testing zone, would have eliminated any possibility of eccentricity in loading, but in lieu of availability, the method arrived at gave reasonably consistent results. It is regrettable that time did not permit the investigation of the two and one-half specimens, however, the testing procedure and apparatus were designed to cover the complete series and it is hoped that this may be done in the future.

OUTLINE OF PROPOSED TESTS

SERIES OF TESTS:

- I. 1/4" max. aggregate
- II. 3/8" max. aggregate
- III. 1/2" max. aggregate

TYPES OF TEST SPECIMENS:

- A. One-bar prism (See Figure 1)
- B. Two-bar prism (See Figure 1)
- C. Three-bar prism (See Figure 1)

VARIATIONS IN SPECIMENS:

- 1. Size of Reinforcing:
 - A. 3/4" bar
 - B. 1" bar
 - C. 1" bar (Square).
- 2. Length of Embedment:
 - A. 12-diameter embedment
 - B. 18-diameter embedment
 - C. 24-diameter embedment
- 3. Clear Bar Spacing and Cover:
 - A. 1.25 X max. size of aggregate
 - B. 1.50 X max. size of aggregate
 - C. 2.00 X max. size of aggregate

NOTES:

- A. Number of Test Specimens:
 - Minimum number of each specimen = 2
 - Total number of specimens for each series = 162
 - Total number of specimens for program = 486
- B. Design Data:
 - Class of concrete = 5000 psi
 - Mix: 1 : 1 1/2 : 1 3/4
 - Reinforcing = Deformed bars.
 - Horizontal pours, Mechanical vibration and "Field"
 - Curing

TESTING OF REINFORCED CONCRETE

LIST OF TESTS:

- I. 1/4 in. diameter
- II. 3/8 in. diameter
- III. 1/2 in. diameter

TYPE OF TEST SPECIMENS:

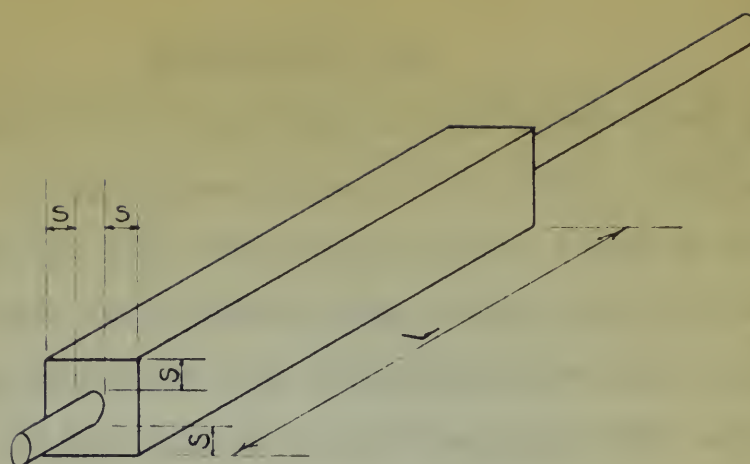
- A. One-bar prism (See Figure 1)
- B. Two-bar prism (See Figure 1)
- C. Three-bar prism (See Figure 1)

VARIATIONS IN SPECIMENS:

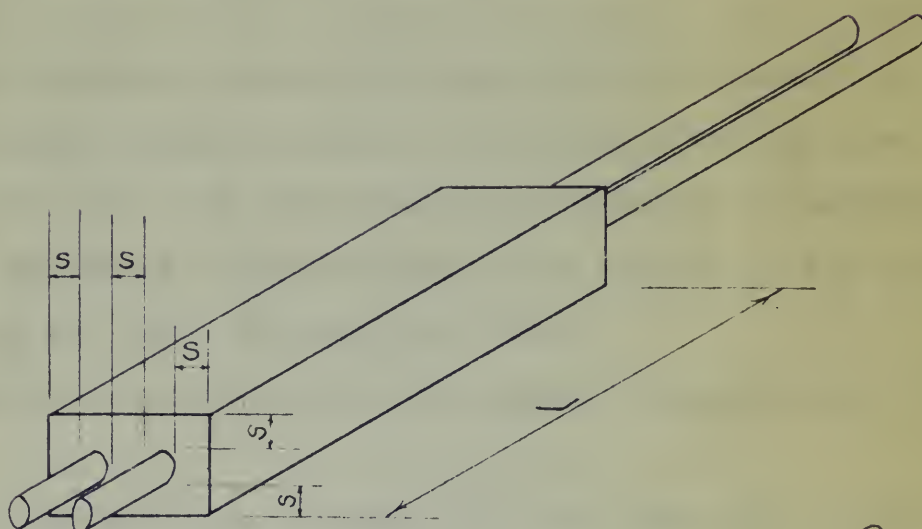
1. Size of Reinforcing:
 - A. 3/4 in. bar
 - B. 1 in. bar
 - C. 1 1/4 in. bar (square)
2. Length of Specimens:
 - A. 12-diameter specimens
 - B. 14-diameter specimens
 - C. 16-diameter specimens
3. Other Bar Spacing and Cover:
 - A. 1.25 X max. size of aggregate
 - B. 1.50 X max. size of aggregate
 - C. 2.00 X max. size of aggregate

NOTES:

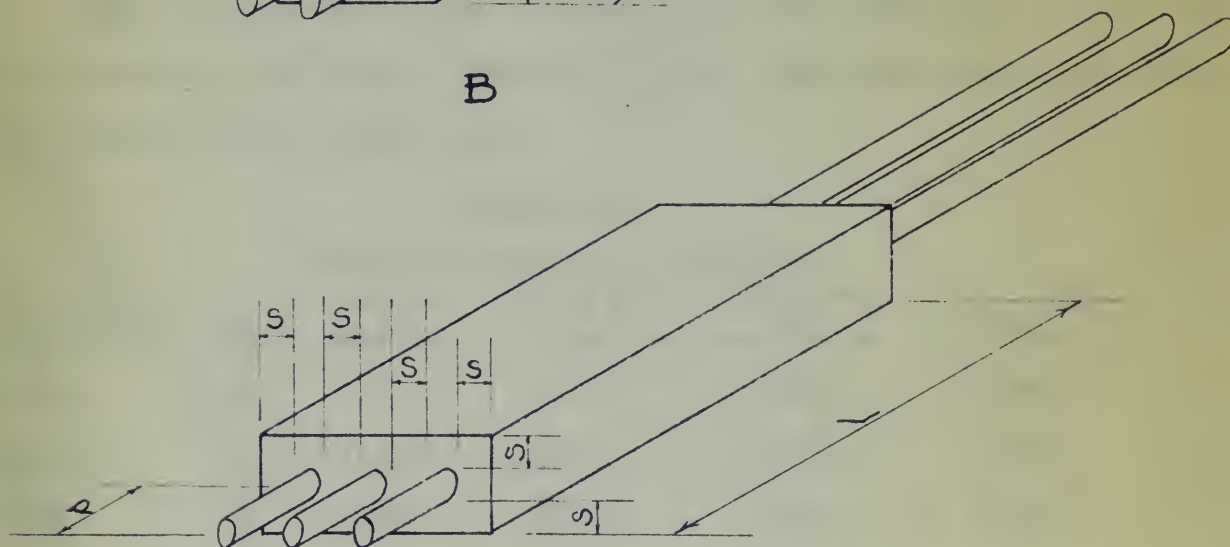
1. Number of Test Specimens:
 - Minimum number of test specimens = 3
 - Total number of specimens for each series = 18
 - Total number of specimens for program = 48
2. Design Data:
 - Class of concrete = 4000 psi
 - Mix: 1 : 1 1/2 : 3 1/4
 - Reinforcing = Deformed bars.
 - Reinforcing bars, mechanical vibration and curing



A



B



C

$P = 1.8"$ at loaded end &
 $1"$ at free end.

S = Clear Spacing & Cover
 L = Length of Embedment

FIG. 1. TEST SPECIMENS

REINFORCING BARS

All deformed bars used were of the "Bamboo Bar" pattern, manufactured by Carnegie-Illinois Steel Corp., intermediate grade, of nominal 3/4- and 1-inch round. Plain bars used were commercial grade cold rolled stock of 3/4 and 1-1/16-inch round.

Diameters and areas were determined from the length-weight measurements of the bars in accordance with ASTM designation A 15-39 and stress calculations were based on the areas thus determined.

The yield point was determined in a 100,000-lb capacity Southwark - Emery testing machine. Load-strain curves were plotted for the bars and the modulus of elasticity so determined was used in computing the correction to be applied to the lower dial readings for slip at the loaded end.

Bar stresses were all within the elastic range for the tests conducted.

The deformed bars were covered with the usual mill scale and had little to no rust. Except as noted the plain bars were clean and smooth with little rust.

Table 1.

PHYSICAL PROPERTIES OF BARS					
<u>Nominal Bar Dia.</u>	<u>Type</u>	<u>Area Sq.In.</u>	<u>Yield Pt. psi.</u>	<u>Ultimate psi.</u>	<u>Mod. Elas. ksi.</u>
1"Ø	d	0.786	41700	67700	29500
3/4"Ø	d	0.442	48800	78000	29500
1"Ø	p	0.885	56800	62500	29600
3/4"Ø	p	0.438	61300	86000	29400

REINFORCING BARS

All delivery bars used were of the "Washed Bar" type, manufactured by Carnegie-Illinois Steel Corp., Intermediate grades, of nominal $\frac{3}{4}$ - and 1-inch round. Plain bars used were commercial grade cold rolled stock of $\frac{3}{4}$ and 1- $\frac{1}{16}$ -inch round. Diameters and areas were determined from the length-weight measurements of the bars in accordance with this designation. A 15-30 and stress calculations were based on the stress bars determined.

The yield point was determined in a 100,000-lb capacity Rockwell - Brinell testing machine. Load-strain curves were plotted for the bars and the modulus of elasticity as determined was used in computing the correction to be applied to the lower dial readings for slip at the loaded end. Bar stresses were all within the elastic range for the tests conducted.

The delivery bars were covered with the usual mill scale and had little to no rust. Except as noted the plain bars were clean and smooth with little rust.

Table 1.

Nominal Bar Dia.	Type	Area Sq. In.	PHYSICAL PROPERTIES OF BARS		Mod. Wt. Lbs.
			Yield Pt. Psi.	Ultimate Psi.	
1" ϕ	A	0.785	41700	57700	29500
$\frac{3}{4}$ " ϕ	A	0.442	48800	78000	29500
1" ϕ	P	0.885	56800	62500	29500
$\frac{3}{4}$ " ϕ	P	0.428	61300	86000	29100

CONCRETE

Concrete was machine mixed for three minutes and proportioned by volume in the ratio 1:1-1/2:1-3/4. The water-cement ratio was 5.0 gal. per sack.

Weight Ratios of Mixes

1/4-in max aggregate	1:1.757:1.940	:0.444 water
3/8-in max aggregate	1:1.757:1.880	on all
1/2-in max aggregate	1:1.750:1.813	

Portland cement meeting the current standard specifications of the ASTM for type III cement was used. (LeHigh-Hi Early).

The coarse aggregate was local limestone, crushed. (See Table 3). The fine aggregate was a Long Island sand known locally as "Cow Bay" sand. (See Table 2).

Table 2.

Sieve Analysis of Sand

<u>U. S. Std. Sieve No.</u>	<u>Percentage Passing by Weight</u>
4	100%
8	97.2
16	78.9
30	49.4
50	11.6
100	1.6

Dry rodded weight 107 lb/cu.ft.

CONCRETE

Concrete was machine mixed for three minutes and incorporated by volume in the ratio 1-1-1/2-3/4. The water-cement ratio was 3.0 gal. per sack.

Weight ratios of mix

1/4-in max aggregate	1:1.75:1.40	17.44% water
3/8-in max aggregate	1:1.75:1.30	on air
1/2-in max aggregate	1:1.75:1.25	

Portland cement meeting the current standard specifications of the ASTM for type III cement was used. (High-strength). The coarse aggregate was local limestone, crushed. (See Table 2). The fine aggregate was a local sand known locally as "Cow Bay" sand. (See Table 2).

Table 2.

Grain Analysis of Sand

Grain No.	U. S. Std.	Percentage Passing
		by Weight
4		100
8		97.2
16		78.9
30		49.4
50		11.6
100		1.6

dry mixed weight 107 lb/cu. ft.

Table 3.Sieve Analysis of Coarse Aggregates

<u>U. S. Std. Sieve No.</u>	<u>Percentages Passing by Weight</u>		
	<u>1/4-in Max</u>	<u>3/8-in Max</u>	<u>1/2-in Max</u>
1/2"	--	--	100%
3/8"	--	100%	88.0
1/4"	100%	--	--
4	91.0	32.9	16.0
8	50.1	--	--
16	11.9	11.6	8.4
30	7.9	--	--
50	6.3	6.6	5.7
100	4.3	3.8	3.7
Dry rodded weight	106 lb/cu.ft.	97.8	95.1
Specific Gravity - 2.72			

Table 4.Average Slump of Concrete Mixes

<u>Max. Agg.</u>	<u>Av. Slump In.</u>
1/4"	0
3/8	1-1/4
1/2	2

Curing of all concrete was in air with the exception of Batch 3-A, which was cured in moist sand for three days. Several compression cylinders were moist-cured for other periods for reasons of comparison.

It is realized that concrete with slumps as low as those obtained in this work cannot be economically placed in practice. Workability can be increased by the use of admixtures.

Table 2.

Slave Analysis of Concrete Aggregates

U. S. Std. Sieve No.	1/4-1/2 Max	3/8-1/2 Max	Percentages Passing by Weight
1/2"	---	---	100%
3/8"	---	100%	97.0
1/4"	100%	---	---
4	91.0	92.9	10.0
8	50.1	---	---
16	11.9	11.6	6.4
30	7.9	---	---
50	6.3	6.6	3.7
100	4.3	3.8	3.7
Dry rounded weight 100 lb = 25.75			
Specific Gravity = 2.75			

Table A.

Average Grains of Concrete Mixes

Max. Size	Average Grains
1/4"	0
3/8"	1-1/4"
1/2"	2

Curing of all concrete was in air with the exception of Batch 3-A, which was cured in moist sand for three days. Several compression cylinders were moist-cured for other periods for reasons of comparison.

It is realized that concrete with slump as low as those obtained in this work cannot be economically placed in practice. Workability can be increased by the use of admixtures.

Six standard 3x6-in cylinders were cast from each batch, except for batch No. 1 for which only three cylinders were cast. These cylinders were tested for compressive strength in accordance with ASTM Designation C 39-44. A summary of the average results of these tests is presented in Figure 2.

six standard 128-in cylinders were used from each batch,
 except for batch No. 1 for which only three cylinders were used.
 These cylinders were tested for compressive strength in
 accordance with ASTM designation C 39-A. A summary of the
 average results of these tests is presented in Figure 2.

Batch	Compressive Strength (psi)
1	10,000
2	10,000
3	10,000

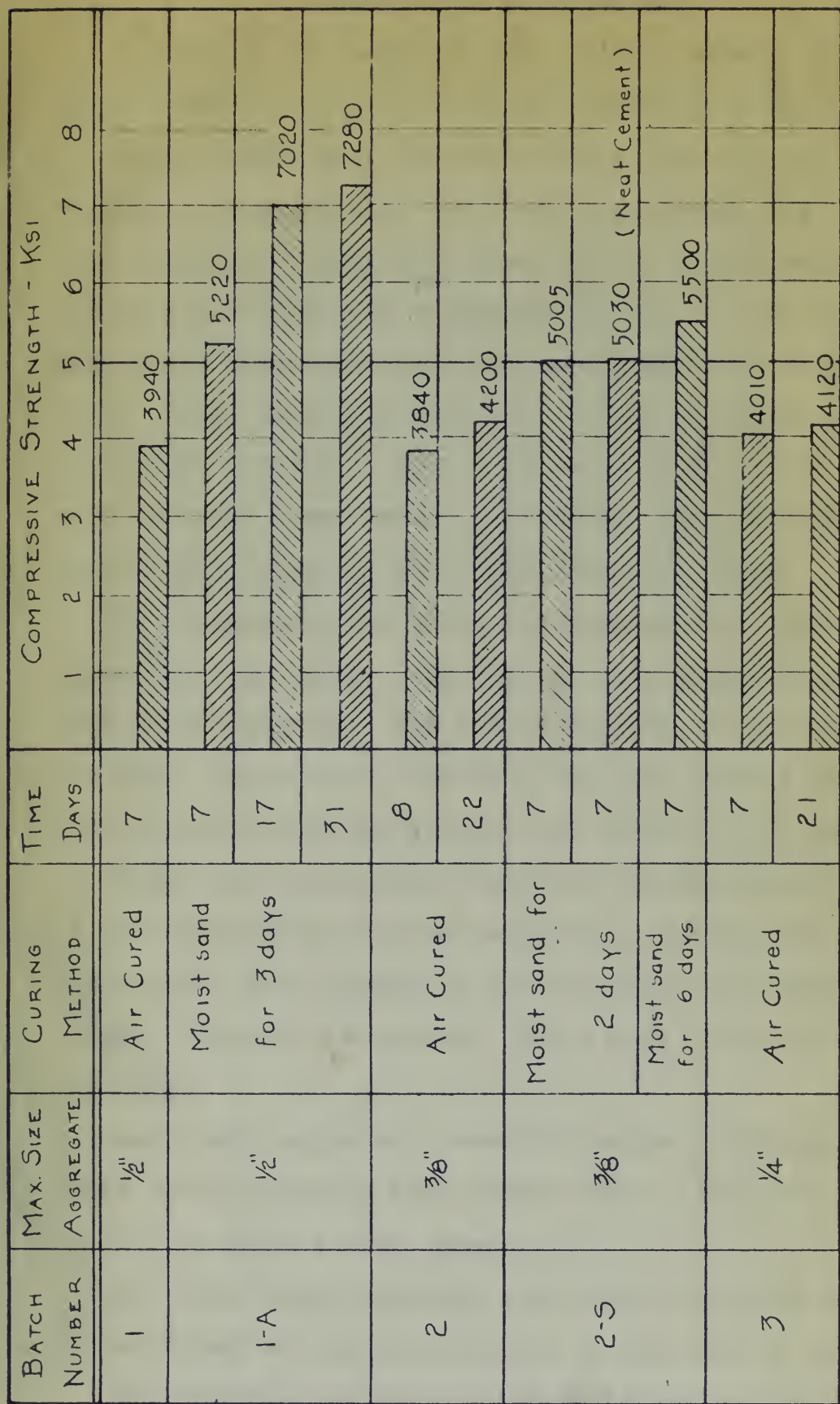


FIG. 2. SUMMARY OF CONCRETE TESTS

Specimen size: Std 3x6 test cylinders. Loading rate: 10,000 #/minute.

All specimens capped on one end only with plaster of paris, except as noted.

DESCRIPTION OF SPECIMENS

A total of 90 specimens were cast in batches of 18 using three reinforcing bars placed as indicated. (Figure 2).

Each "batch" cast employed either 1/2, 3/8, or 1/4-inch maximum size aggregate, the clear bar spacing and cover varying as a function of aggregate size. Thus a batch consisted of 9 1-in round bars with embedments of 12, 18, and 24 inches, and with three different covers for each embedment varying from 5/16" to 1", depending upon the size of aggregate employed; and 9 3/4-inch round bars with embedments of 9, 13-1/2, and 18 inches with the same cover as the 1-in bars. Six standard 3x6-inch compression cylinders were cast from each batch.

To accommodate the variables required per batch, forms were constructed by using a set size of base plate and side plates of 3/4" -5 ply plywood. End pieces were the controlling section of the form. These were accurately cut and drilled from 3/4" plywood to conform to the spacing and cover for the batch.

Forms were assembled by setting the end pieces on the base at the required spacing for embedment, placing the side pieces and wedging them solidly to the end pieces by means of wooden wedges. (Figures 3-A and-B). Forms were oiled prior to placing the steel.

Loose mill scale was removed from the reinforcing bars and a short section wrapped with rubber tape in order to secure a snug fit in the holes in the forms.

The forms thus assembled were easily stripped and possessed the advantage of being adjustable to any size of specimen.

The concrete was hand placed in layers and well rodded to insure dense specimens free from "honeycombing" and air pockets.

DESCRIPTION OF SPECIMENS

A total of 90 specimens were cast in batches of 12 using three reinforcing bars placed as indicated. (Figure 2).

Each "batch" cast weighed either 1 1/2, 3/4, or 1 1/4 tons. Specimens also weighed, the clear bar spacing and cover varying as a function of aggregate size. Thus a batch consisted of 9 1-in round bars with embedments of 12, 18, and 24 inches, and with three different covers for each embedment varying from 2 1/2" to 1", depending upon the size of aggregate employed; and 9 3/4-inch round bars with embedments of 9, 12-1/2, and 18 inches with the same cover as the 1-in bars. Six standard two-inch compression cylinders were cast from each batch.

To accommodate the variable rebar and bed, specimens constructed by using a set size of base plate and side plates of 3/4" - 2 ply plywood. And pieces were the controlling section of the form. These were accurately cut and drilled from 3/4" ply wood to conform to the spacing and cover for the batch.

Forms were assembled by setting the end plates on the base at the required spacing for embedment, placing the side plates and wedging them solidly to the end plates by means of wooden wedges. (Figures 3-A and 3-B). Forms were dried prior to placing the steel.

Loose oil seals was removed from the reinforcing bars and a short section wrapped with rubber tape in order to secure a snug fit in the holes in the forms.

The forms thus assembled were easily stripped and reassembled the advantage of being adjustable to any size of specimen.

The concrete was then placed in layers and well tamped to insure dense specimens free from "bleeding" and air pockets.

The top was screeded off to the height of the end forms, and the form vibrated from one to three minutes until a water sheen appeared on the surface.

The boy was scolded all the while of the two boys, and
 the two whispered from one to the other until a man
 appeared on the scene.

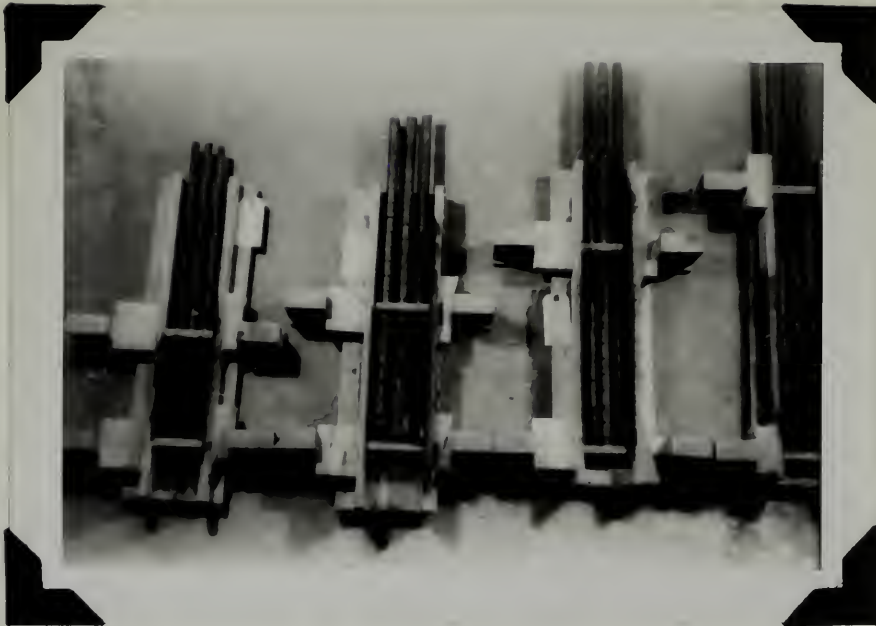


Figure No. 3-A.

Type of form used for all specimens showing placement of reinforcing bars. End blocks control the cover and clear bar spacing. Note wedges that take side pieces up solid.



Figure No. 3-A.

Type of form used for all specimens showing placement of reinforcing bars, and blocks control the cover and clear bar spacing. Note wedges that take side plates up solid.



Figure 3-B.

Forms ready for placing concrete. A "batch" consisted of 18 specimens and 6 3x6-inch compression cylinders.



Figure 3-B.

Forms ready for placing concrete. A "bore" consisted of 18 specimens and 8 3/4-inch compression cylinders.

The vibrator consisted of a spring-mounted table with a device for clamping down the form. This table was vibrated by a 1/2 H.P. motor which was connected to the table by an eccentric. This arrangement was used to vibrate all specimens, the weight varying from 8 to 70 lbs. (See Figure 4).

Forms were stripped at the end of 24 hours. (Figure 3-C). The specimens were cured for seven days before testing. Batch 3-A was cured for 3 days in moist sand; all other batches were air cured at 55-65F, no attempt being made to control the moisture. This was done in an attempt to approach the field conditions of curing precast concrete.

Table 5.

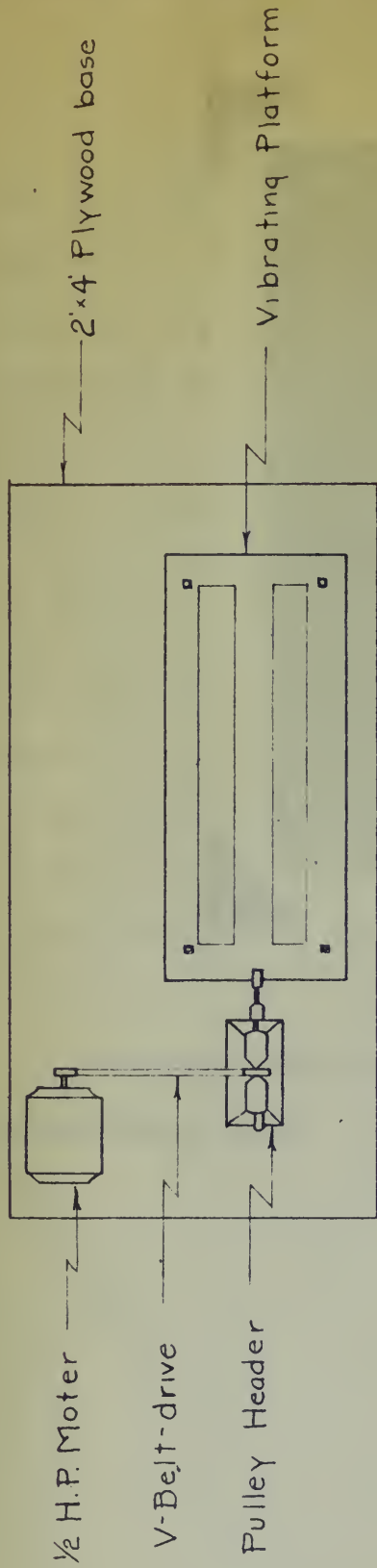
SPECIMENS CAST

<u>Batch</u>	<u>7-day comp. strength-psi</u>	<u>Bars</u>	<u>Embedment in.</u>	<u>Clear Spacing & Cover in.</u>	<u>No. of Specs.</u>
1	3940	1"Ød 3/4"Ød	12, 18, 24 9, 13½, 18	1, 3/4, 5/8	9 9 <u>18</u>
1-A	5220	do	do	do	18
2	3840	do	do	3/4, 9/16, 15/32	18
3	4010	do	do	5/16, 3/8, 1/2	18
2-S	5005	1"Øp 3/4"Øp	do do	11/16, 1/2, 13/32 3/4, 9/16, 15/16	9 9 <u>18</u>
Total					90

The vibrator consisted of a spring-mounted table with a device for clamping down the form. This table was vibrated by a 1/2 H.P. motor which was connected to the table by an eccentric. This arrangement was used to vibrate all specimens, the weight varying from 2 to 70 lbs. (See Figure 4). Forms were stripped at the end of 24 hours. (Figure 3-C). The specimens were cured for seven days before testing. Batch 3-A was cured for 3 days in moist sand; all other specimens were air cured at 55-65%, no attempt being made to control the moisture. This was done in an attempt to reproduce the field conditions of curing precast concrete.

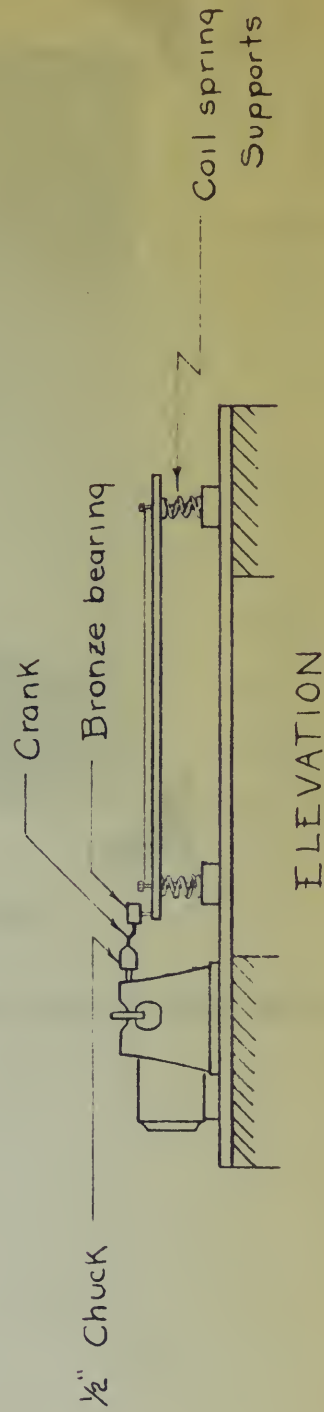
Table 1.
BATCHING CAST

Batch	7-day comp. strength-psi	Batch	28-day strength-psi	Clear spacing & cover in.	No. of Spec.
1	3940	1000	11,11,11	1 1/2 x 3 1/8	9 2 16
1-A	3230	do	do	do	16
2	3840	do	do	1 1/2 x 3 1/8, 1 1/2 x 3 1/8	16
3	4010	do	do	1 1/2 x 3 1/8, 1 1/2	16
3-B	3000	1000	11,11,11	1 1/2 x 3 1/8, 1 1/2 x 3 1/8	9 2 16
Total					90



PLAN

Scale 1"=1'0"



ELEVATION

FIG. 4. SKETCH OF VIBRATOR



Figure No. 3-C.

A "batch" prior to welding the lugs on the ends of the reinforcing bars.



Figure No. 3-6.

A "base" prior to welding the legs on the ends of the

reinforcing bars.

DESCRIPTION OF TESTING APPARATUS

TESTING YOKES

The testing of 3-bar specimens required apparatus particularly adapted to the purpose. This consisted of two rectangular yokes similar in detail except for the length of vertical members. All crossheads were of two 1x5x1'-6" cold rolled steel separated by the vertical members, two 5/8 x 4-in. bars with a 1/4-in spacer between. Each joint was pin-connected with a 1-1/2-in bolt.

Each of the two yokes was secured to its respective crosshead in the testing machine by a 2-1/2x1'-5" bolt, the shank milled on two sides to 1-7/16-in thickness to permit fitting between the bars of the yoke crosshead. (See Figures Nos. 5 and 6).

Design load, based on a maximum possible resistance of specimens, was 100,000 lb. Design deflection for the crossheads was a practical minimum.

DESCRIPTION OF TESTING APPARATUS

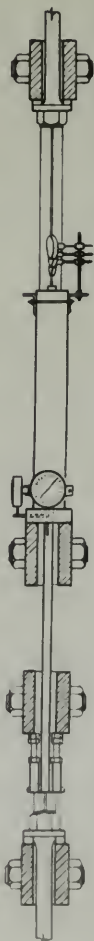
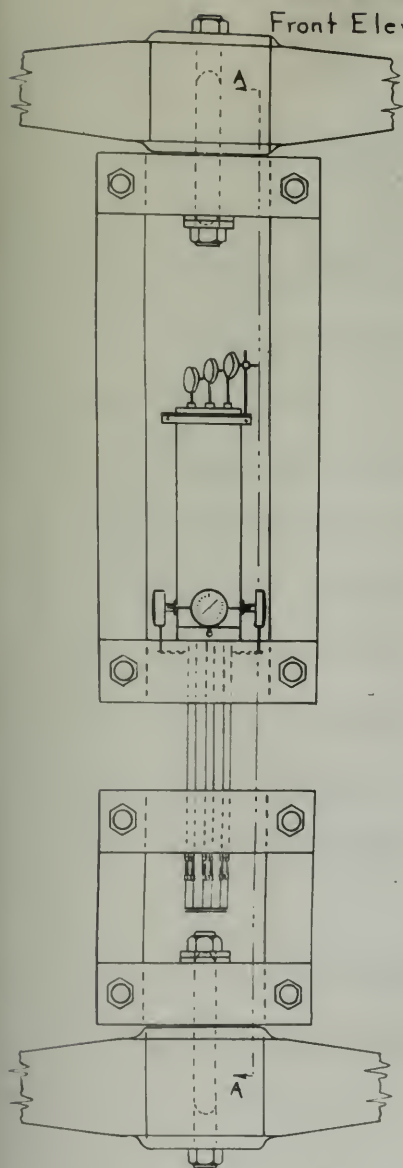
TESTING STAND

The testing of 3-year specimens required an apparatus particularly adapted to the purpose. This consisted of two rectangular frames standing in parallel spaced for the length of vertical members. All crossmembers were of two 1x2x1/2-in. cold rolled steel supported by two vertical members, two 3/4 x 1/2-in. bars with a 1/4-in. square between. Each joint was pre-stressed with a 1-1/2-in. bolt.

Each of the two frames was anchored to the supporting structure in the testing machine by a 2-1/2x1-1/2 bolt, the chains hinged on two sides to 1-7/8-in. linkages to permit tilting between the bars of the joint crossmembers. (See Figure Nos. 3 and 4). Better load, based on a maximum possible resistance of specimens, was 100,000 lb. Design deflection for the specimens was a practical minimum.

SET-UP FOR TESTING

Front Elevation

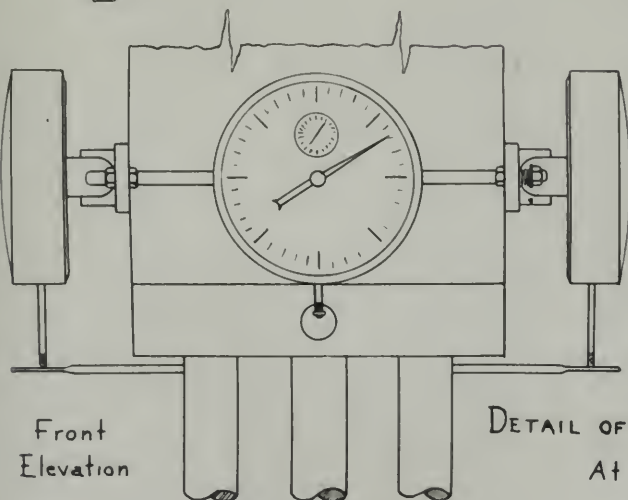
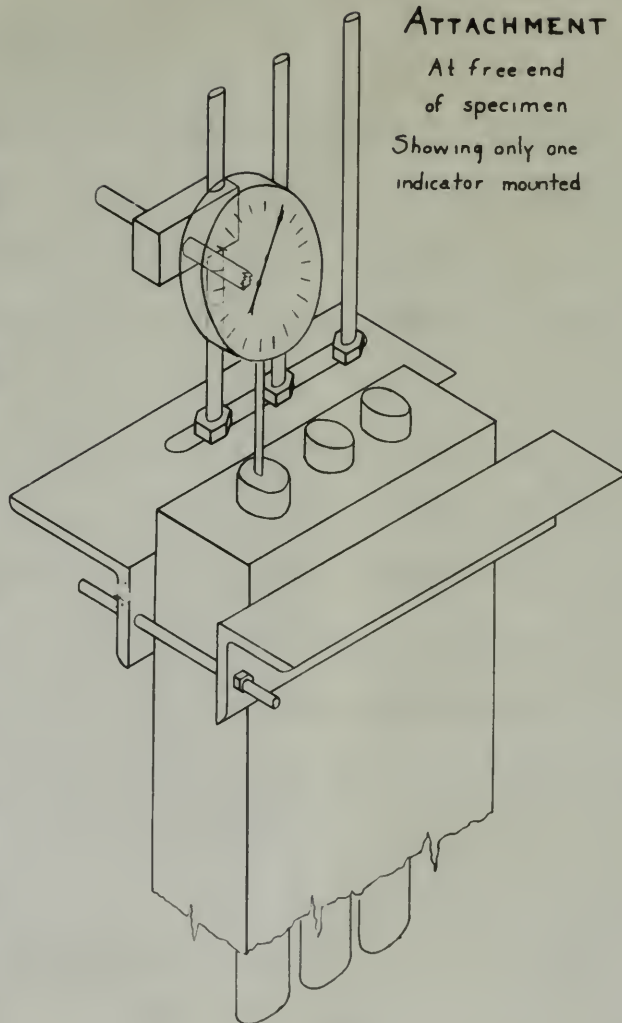
Scale: $1" = \frac{1}{4}"$ 

Section at A-A

DETAIL OF DIAL

ATTACHMENT

At free end
of specimen
Showing only one
indicator mounted

Front
Elevation

DETAIL OF DIAL ATTACHMENT

At loaded end

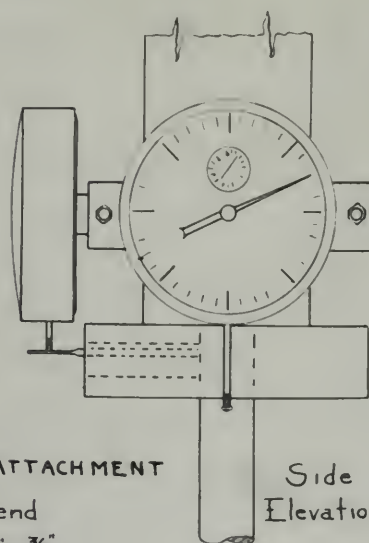
Scale: $1" = \frac{3}{8}"$ Side
Elevation

FIG. 5 TESTING RIG

YOKE WITH DETAILS OF DIAL INDICATOR ARRANGEMENT
AND ATTACHMENT

BEARING PLATES

Steel bearing plates of 1x3-in stock were used to transmit the load from the base of the specimen to the crosshead of the upper yoke. To accommodate the variable clear bar spacing and the casting schedule, fabrication of fifty-four bearing plates was required.

Bearing plates were all drilled with 7/16-in holes to accommodate the final arrangement for slip measurement at the loaded end. Those 5-1/2-in long and less were drilled from the edge to the center hole; those over 5-1/2-in long, because of clearance restrictions between the edge of the specimen and the vertical members of the yoke, required two additional holes on the opposite side, drilled to the end holes.

Table 6.BEARING PLATE SCHEDULE

<u>No. Req'd.</u>	<u>Pl. No.</u>	<u>L"</u>	<u>C"</u>	<u>D"</u>
3	1	4.0	1-1/16 1.0625	7/8"
3	2	4.0	1-1/8 1.125	7/8"
3	3	4.5	1-1/4 1.250	1"
3	4	4.5	1-7/32 1.21875	1"
3	5	4.5	1-5/16 1.3125	1"
3	6	4.5	1-5/16 1.3125	1-1/4"
3	7	4.5	1-3/8 1.375	1-1/4"
3	8	5.0	1-3/8 1.375	1"
3	9	5.0	1-1/2 1.50	1-1/4"
3	10	5.0	1-15/32 1.46875	1-1/4"
6	11	5.5	1-1/2 1.50	1"
3	12	5.5	1-9/16 1.5625	1-1/4"

HEARING PLATES

Small hearing plates of 1 1/2-in stock were used to transmit the load from the base of the specimen to the crosshead of the test frame. To accommodate the variable clearances between the hearing plates, the hearing schedule, fabrication of fifty-four hearing plates was required.

Hearing plates were all drilled with 7/16-in holes to accommodate the final arrangement for slip measurement of the loaded end. Those 7-1 1/2-in long and those were drilled from the edge to the center holes; those over 5-1/2-in long, because of clearance restrictions between the edge of the specimen and the vertical members of the frame, required two additional holes on the opposite side, drilled to the same holes.

Table 2.

HEARING PLATE SCHEDULE

No. Holes	Length, in.	Weight, lb.	Drill Size
3	1	1.0625	1-1/16
3	2	1.125	1-1/8
3	3	1.250	1-1/4
3	4	1.21875	1-7/32
3	5	1.3125	1-5/16
3	6	1.3125	1-5/16
3	7	1.375	1-3/8
3	8	1.375	1-3/8
3	9	1.50	1-1/2
3	10	1.46875	1-13/32
4	11	1.50	1-1/2
3	12	1.5625	1-9/16

Table 6. (Continued)BEARING PLATE SCHEDULE

<u>No. Req'd.</u>	<u>Pl. No.</u>	<u>L"</u>	<u>C"</u>	<u>D"</u>
3	13	5.5	1-5/8 1.625	1-1/4"
6	14	6.0	1-3/4 1.75	1-1/4"
3	15	7.0	1-3/4 1.75	1"
3	16	7.0	2 2.00	1-1/4"

L = Length

C = Ctr. to Ctr. bar spacing

D = Diameter of drilled hole

DIAL ARRANGEMENT AND ATTACHMENT

Illustrations of the method used for measuring slip at the loaded and free ends are shown in Figures Nos. 5 and 6 and described under "Testing Procedure".

Table 6. (Continued)

BEARING RATE SCHEDULE

Test No.	Test No.	Test No.	Test No.	Test No.
13	1-2-4	1-2-5	1-2-6	1-2-7
14	1-2-8	1-2-9	1-2-10	1-2-11
15	1-2-12	1-2-13	1-2-14	1-2-15
16	1-2-16	1-2-17	1-2-18	1-2-19

DIAL ARRANGEMENT AND ATTACHMENT

Illustrations of the method used for measuring life of the loaded and free ends are shown in Figure Nos. 5 and 6 and described under "Testing Procedure".

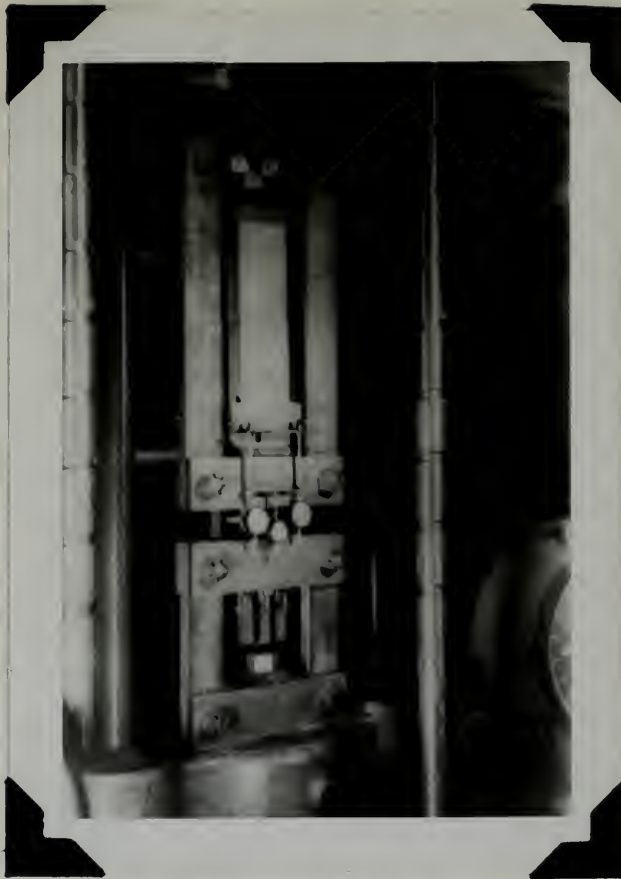


Figure No. 6.

The arrangement of the testing yokes with a 24 inch embedment specimen, and the dial arrangement. Dial rigging is constructed so that it is adjustable to any size specimen.

The 1-inch bearing plate rides on the upper yoke. The pull is transmitted through the bolt columns to the welded lugs on the ends of the reinforcing bars. Yokes were bolted to the crossheads of the testing machine.

The testing rig was left mounted in the testing machine throughout the experiment. Specimens were mounted by removing the crosshead members from the upper and lower yokes.

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Figure No. 5.

The arrangement of the testing yokes with a 2A test specimen, specimen, and the dial arrangement. Dial rigging is considered so that it is adjustable to any size specimen.

The 2-inch bearing plate rides on the upper yoke. The ball is transmitted through the hole column to the solid legs on the ends of the rotating bars. These were bolted to the crossheads of the testing machine.

The testing rig was left mounted in the testing machine throughout the experiment. Specimens were mounted by removing the crosshead members from the upper and lower yokes.

PREPARATION OF SPECIMENS FOR TEST

The specimens, after curing 5 days, were prepared as required for the testing procedure adopted.

Bearing plates were slipped onto the reinforcing bars. Three-inch lengths of $3/4$ -in round bars were welded, two per reinforcing bar, to the loading-end in positions such that three of these "lugs" projected on each side of the plane of reinforcing bars. The "upper" ends of these lugs were machined prior to placing and in welding them on, all six faces were maintained as nearly as possible in a plane perpendicular to the specimen axis.

It was found necessary to weld light steel rods across the three lugs on each side to prevent any tendency of the loaded bar ends to move with respect to each other during loading.

With the base plate snug against the concrete, short $5/32$ -in rods were spot welded to the reinforcing bars at right angles through the holes in the base plates. On specimens $5-1/2$ -in and less the rods were welded on just below the bearing plate and at right angles to the center rod. (Figure 5).

These rods were flat and smooth on the free end and oriented with the flat sides parallel to the bearing plate. It was on these surfaces that the tips of the dial gages rested during testing.

The bearing end of the concrete was not capped because the plywood forms left a very flat and smooth surface which was perpendicular to the axis of the reinforcing bars.

PREPARATION OF SPECIMENS FOR TEST

The specimens, after curing 5 days, were prepared as required for the testing procedure adopted.

Bearing plates were slipped onto the reinforcing bars.

Three-inch lengths of $\frac{3}{4}$ -in round bars were welded, two per reinforcing bar, to the loading-end in positions such that some of these "lips" projected on each side of the plane of reinforcing bars. The "upper" ends of these lips were machined prior to

placing and in welding them on, all air leaves were maintained as nearly as possible in a plane perpendicular to the specimen axis.

It was found necessary to weld light steel rods across the

three lips on each side to prevent any tendency of the loaded

bar ends to move with respect to each other during loading.

With the base plate snug against the concrete, about $\frac{1}{16}$ -in

rods were spot welded to the reinforcing bars at right angles

through the holes in the base plates. On specimens $2-1\frac{1}{2}$ -in and

less the rods were welded on just before the bearing plates and at

right angles to the center rods. (Figure 2).

These rods were filed and smoothed on the top and end surfaces

with the flat sides parallel to the bearing plates. It was on

these surfaces that the tips of the dial gages rested during

testing.

The bottom end of the concrete was not capped because the

typical form left a very flat and smooth surface which was

perpendicular to the axis of the reinforcing bars.

TESTING PROCEDURE

The specimens were tested in a 100,000 lb capacity fluid support Southwark-Emery testing machine, and the load was applied at the rate of about 10,000 lb/min., the machine being stopped to read the dial gages. The specimen was seated on a 1-in bearing block with holes of 1/4-in greater diameter than the steel bars bored to the individual spacing of the specimen. Such economy was required to obtain a maximum area of concrete in compression. The base plates were supported by the lower crosshead of the upper yoke.

Slips of the bars were measured by 1/10,000-in Federal dial gages on the unloaded end, and by 1/1,000 dial gages on the loaded end. Readings of the lower dials were estimated to 2/10,000 inch.

At the unloaded end the dials were mounted on brackets adjustable horizontally, radially, and vertically, the support being two angles firmly clamped to the upper end of the concrete by two spring-loaded bolts. (Figure No. 5). To obtain clearance between dials, a 1-in tip extension was employed on the center dial.

The very limited space for placing dials due to the height limitations of the testing machine and the very close spacing of the bars led to several different dial arrangements at the loaded end. First the dials were mounted on the bars by brackets, the tips resting on a flat horizontal bar supported by the lower end of the concrete. (Figures Nos. 6 and 7.). This arrangement proved to be unsatisfactory.

TESTING PROCEDURE

The specimens were tested in a 100,000 lb capacity test

support. A constant load was applied at the rate of about 10,000 lb/min., the specimen being stopped to read the dial gages. The specimen was tested on a 1-in bearing block with holes of 1/4-in diameter spaced 7 in apart. The axial force was the resultant spacing of the specimen. Such accuracy was required to obtain a uniform area of compression. The base plates were supported by the lower crosshead of the upper frame.

Slips of the bars were measured by 1/10,000-in dial gages on the unloaded end, and by 1/1,000 dial gages on the loaded end. Readings of the lower dial were estimated to 1/10,000 inch. At the unloaded end the dial was mounted on a vertical support. The specimen being tested horizontally, vertically, and vertically, the support being two gages fixedly clamped to the upper end of the concrete by two spring-loaded bolts. (Figure No. 2). To obtain clearance between dials, a 1-in tip extension was attached on the lower dial.

The very limited space for placing dial was to the height limitations of the testing machine and the very close spacing of the bars led to several different dial arrangements at the loaded end. First the dial was mounted on the bar by two bolts, the tips resting on a flat horizontal bar supported by two lower ends of the concrete. (Figure Nos. 3 and 4). This arrangement proved to be unsatisfactory.

A second arrangement (Figure No. 5), placed the dials on a bracket mounted on the lower end of the concrete about 2-in above the compression face, the tips resting on the projecting rods which are described under "Preparation of Specimen for Test".

When of significant magnitude, the slip at the loaded end as read on the dial gages was corrected for elongation of the projecting steel. Slip at the free end was read directly from the dial gages.

A second attachment (Figure No. 2) joined the skin of a
 ventral winged on the lower end of the opposite wing 2-11 above
 the humeral line, the line joining on the opposite side
 which are attached under "prothoracic of abdomen for 1st".
 Some of abdominal segments, the size of the segments and as
 read on the dial pages was corrected for elongation of the seg-
 menting process. Size of the feet was read directly from the
 dial pages.

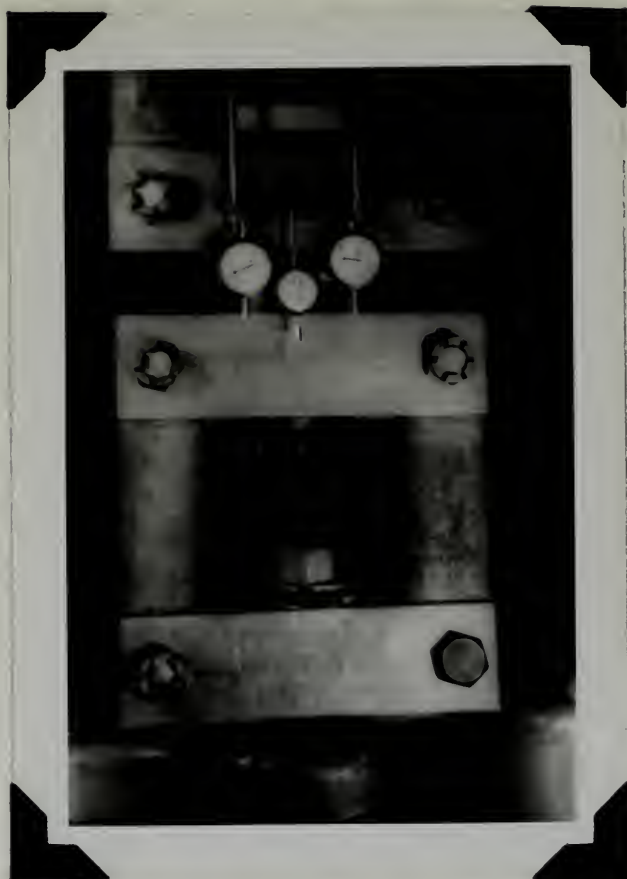


Figure No. 7.

The lower yoke showing initial dial arrangement for measuring slip at the loaded end. Dials were clamped to reinforcing bars and registered movement at the lower end of the concrete through the extension. This method was changed to a more satisfactory arrangement by drilling holes in the bearing plate and welding "tips" to the reinforcing bar. Dials clamped to the concrete rode on the "tips" thus measuring the motion directly at the base of the specimen.



Figure No. 2.

The lower yoke showing initial distal arrangement for measuring slip at the loaded end. Galls were cleaned to relieving bars and registered movement at the lower end at the separate through the extension. This motion was changed to a more satisfactory arrangement by adding noise in the bearing plate and welding "rings" to the relieving bar. A plate attached to the concrete rods on the "rings" thus measuring the motion directly at the base of the specimen.

Initial dial-gage readings were taken at a machine load of 2,000 lb., then at 5,000 lb., and readings at increments of 2500 lbs. were taken until failure. All dial gages were left in place until failure.

The problem of producing equal motion in the loaded ends of the three bars was solved by the use of the lugs as described in "Preparation of Specimens for Test". Load was applied to the bars through short columns in the form of six 5/8x1-1/4-in tap screws with double nuts, the head of each tap screw bearing on the machined surface of a lug and the nut bearing on the lower face of the upper crosshead of the lower yoke. (Figure No. 5).

In placing the tap screws, minimum clearance was used to make the nuts take the load and all nuts were taken up finger-tight. The machine was loaded to 2,000 lb. by hand and all dials placed.

As a check on the equality of loading obtained in the three bars, Porter-Lipp strain gages were placed on the bars in a number of tests.

Initial trial-logs were taken at a machine load of

2,000 lbs., then at 2,500 lbs., and finally at intermediate of 3,000 lbs. with bars until failure. All trial logs were left in place until failure.

The purpose of producing equal action in the loaded ends of the three bars was solved by the use of the logs as described in "Preparation of Specimens for Test". Load was applied to the bars through short columns in the form of six 3/4-in x 1-in x 1-in bars with double ends. The head of each bar was bearing on the machined surface of a log and the bar resting on the lower face of the upper crosshead of the lower yoke. (Figure No. 3). In placing the log across, minimum clearance was used to make the nuts take the load and all nuts were turned up (tight) slightly. The machine was loaded to 2,000 lbs. by leads and all trials placed.

As a check on the equality of loading obtained in the three bars, Porter-type strain gauges were placed on the bars in a number of cases.

The results of the tests were as follows: The first trial was made at a load of 2,000 lbs. and the bars failed at a load of 2,500 lbs. The second trial was made at a load of 2,500 lbs. and the bars failed at a load of 3,000 lbs. The third trial was made at a load of 3,000 lbs. and the bars failed at a load of 3,500 lbs. The fourth trial was made at a load of 3,500 lbs. and the bars failed at a load of 4,000 lbs. The fifth trial was made at a load of 4,000 lbs. and the bars failed at a load of 4,500 lbs. The sixth trial was made at a load of 4,500 lbs. and the bars failed at a load of 5,000 lbs. The seventh trial was made at a load of 5,000 lbs. and the bars failed at a load of 5,500 lbs. The eighth trial was made at a load of 5,500 lbs. and the bars failed at a load of 6,000 lbs. The ninth trial was made at a load of 6,000 lbs. and the bars failed at a load of 6,500 lbs. The tenth trial was made at a load of 6,500 lbs. and the bars failed at a load of 7,000 lbs. The eleventh trial was made at a load of 7,000 lbs. and the bars failed at a load of 7,500 lbs. The twelfth trial was made at a load of 7,500 lbs. and the bars failed at a load of 8,000 lbs. The thirteenth trial was made at a load of 8,000 lbs. and the bars failed at a load of 8,500 lbs. The fourteenth trial was made at a load of 8,500 lbs. and the bars failed at a load of 9,000 lbs. The fifteenth trial was made at a load of 9,000 lbs. and the bars failed at a load of 9,500 lbs. The sixteenth trial was made at a load of 9,500 lbs. and the bars failed at a load of 10,000 lbs. The seventeenth trial was made at a load of 10,000 lbs. and the bars failed at a load of 10,500 lbs. The eighteenth trial was made at a load of 10,500 lbs. and the bars failed at a load of 11,000 lbs. The nineteenth trial was made at a load of 11,000 lbs. and the bars failed at a load of 11,500 lbs. The twentieth trial was made at a load of 11,500 lbs. and the bars failed at a load of 12,000 lbs. The twenty-first trial was made at a load of 12,000 lbs. and the bars failed at a load of 12,500 lbs. The twenty-second trial was made at a load of 12,500 lbs. and the bars failed at a load of 13,000 lbs. The twenty-third trial was made at a load of 13,000 lbs. and the bars failed at a load of 13,500 lbs. The twenty-fourth trial was made at a load of 13,500 lbs. and the bars failed at a load of 14,000 lbs. The twenty-fifth trial was made at a load of 14,000 lbs. and the bars failed at a load of 14,500 lbs. The twenty-sixth trial was made at a load of 14,500 lbs. and the bars failed at a load of 15,000 lbs. The twenty-seventh trial was made at a load of 15,000 lbs. and the bars failed at a load of 15,500 lbs. The twenty-eighth trial was made at a load of 15,500 lbs. and the bars failed at a load of 16,000 lbs. The twenty-ninth trial was made at a load of 16,000 lbs. and the bars failed at a load of 16,500 lbs. The thirtieth trial was made at a load of 16,500 lbs. and the bars failed at a load of 17,000 lbs. The thirty-first trial was made at a load of 17,000 lbs. and the bars failed at a load of 17,500 lbs. The thirty-second trial was made at a load of 17,500 lbs. and the bars failed at a load of 18,000 lbs. The thirty-third trial was made at a load of 18,000 lbs. and the bars failed at a load of 18,500 lbs. The thirty-fourth trial was made at a load of 18,500 lbs. and the bars failed at a load of 19,000 lbs. The thirty-fifth trial was made at a load of 19,000 lbs. and the bars failed at a load of 19,500 lbs. The thirty-sixth trial was made at a load of 19,500 lbs. and the bars failed at a load of 20,000 lbs. The thirty-seventh trial was made at a load of 20,000 lbs. and the bars failed at a load of 20,500 lbs. The thirty-eighth trial was made at a load of 20,500 lbs. and the bars failed at a load of 21,000 lbs. The thirty-ninth trial was made at a load of 21,000 lbs. and the bars failed at a load of 21,500 lbs. The fortieth trial was made at a load of 21,500 lbs. and the bars failed at a load of 22,000 lbs. The forty-first trial was made at a load of 22,000 lbs. and the bars failed at a load of 22,500 lbs. The forty-second trial was made at a load of 22,500 lbs. and the bars failed at a load of 23,000 lbs. The forty-third trial was made at a load of 23,000 lbs. and the bars failed at a load of 23,500 lbs. The forty-fourth trial was made at a load of 23,500 lbs. and the bars failed at a load of 24,000 lbs. The forty-fifth trial was made at a load of 24,000 lbs. and the bars failed at a load of 24,500 lbs. The forty-sixth trial was made at a load of 24,500 lbs. and the bars failed at a load of 25,000 lbs. The forty-seventh trial was made at a load of 25,000 lbs. and the bars failed at a load of 25,500 lbs. The forty-eighth trial was made at a load of 25,500 lbs. and the bars failed at a load of 26,000 lbs. The forty-ninth trial was made at a load of 26,000 lbs. and the bars failed at a load of 26,500 lbs. The fiftieth trial was made at a load of 26,500 lbs. and the bars failed at a load of 27,000 lbs. The fifty-first trial was made at a load of 27,000 lbs. and the bars failed at a load of 27,500 lbs. The fifty-second trial was made at a load of 27,500 lbs. and the bars failed at a load of 28,000 lbs. The fifty-third trial was made at a load of 28,000 lbs. and the bars failed at a load of 28,500 lbs. The fifty-fourth trial was made at a load of 28,500 lbs. and the bars failed at a load of 29,000 lbs. The fifty-fifth trial was made at a load of 29,000 lbs. and the bars failed at a load of 29,500 lbs. The fifty-sixth trial was made at a load of 29,500 lbs. and the bars failed at a load of 30,000 lbs. The fifty-seventh trial was made at a load of 30,000 lbs. and the bars failed at a load of 30,500 lbs. The fifty-eighth trial was made at a load of 30,500 lbs. and the bars failed at a load of 31,000 lbs. The fifty-ninth trial was made at a load of 31,000 lbs. and the bars failed at a load of 31,500 lbs. The sixtieth trial was made at a load of 31,500 lbs. and the bars failed at a load of 32,000 lbs. The sixty-first trial was made at a load of 32,000 lbs. and the bars failed at a load of 32,500 lbs. The sixty-second trial was made at a load of 32,500 lbs. and the bars failed at a load of 33,000 lbs. The sixty-third trial was made at a load of 33,000 lbs. and the bars failed at a load of 33,500 lbs. The sixty-fourth trial was made at a load of 33,500 lbs. and the bars failed at a load of 34,000 lbs. The sixty-fifth trial was made at a load of 34,000 lbs. and the bars failed at a load of 34,500 lbs. The sixty-sixth trial was made at a load of 34,500 lbs. and the bars failed at a load of 35,000 lbs. The sixty-seventh trial was made at a load of 35,000 lbs. and the bars failed at a load of 35,500 lbs. The sixty-eighth trial was made at a load of 35,500 lbs. and the bars failed at a load of 36,000 lbs. The sixty-ninth trial was made at a load of 36,000 lbs. and the bars failed at a load of 36,500 lbs. The seventieth trial was made at a load of 36,500 lbs. and the bars failed at a load of 37,000 lbs. The seventy-first trial was made at a load of 37,000 lbs. and the bars failed at a load of 37,500 lbs. The seventy-second trial was made at a load of 37,500 lbs. and the bars failed at a load of 38,000 lbs. The seventy-third trial was made at a load of 38,000 lbs. and the bars failed at a load of 38,500 lbs. The seventy-fourth trial was made at a load of 38,500 lbs. and the bars failed at a load of 39,000 lbs. The seventy-fifth trial was made at a load of 39,000 lbs. and the bars failed at a load of 39,500 lbs. The seventy-sixth trial was made at a load of 39,500 lbs. and the bars failed at a load of 40,000 lbs. The seventy-seventh trial was made at a load of 40,000 lbs. and the bars failed at a load of 40,500 lbs. The seventy-eighth trial was made at a load of 40,500 lbs. and the bars failed at a load of 41,000 lbs. The seventy-ninth trial was made at a load of 41,000 lbs. and the bars failed at a load of 41,500 lbs. The eightieth trial was made at a load of 41,500 lbs. and the bars failed at a load of 42,000 lbs. The eighty-first trial was made at a load of 42,000 lbs. and the bars failed at a load of 42,500 lbs. The eighty-second trial was made at a load of 42,500 lbs. and the bars failed at a load of 43,000 lbs. The eighty-third trial was made at a load of 43,000 lbs. and the bars failed at a load of 43,500 lbs. The eighty-fourth trial was made at a load of 43,500 lbs. and the bars failed at a load of 44,000 lbs. The eighty-fifth trial was made at a load of 44,000 lbs. and the bars failed at a load of 44,500 lbs. The eighty-sixth trial was made at a load of 44,500 lbs. and the bars failed at a load of 45,000 lbs. The eighty-seventh trial was made at a load of 45,000 lbs. and the bars failed at a load of 45,500 lbs. The eighty-eighth trial was made at a load of 45,500 lbs. and the bars failed at a load of 46,000 lbs. The eighty-ninth trial was made at a load of 46,000 lbs. and the bars failed at a load of 46,500 lbs. The ninetieth trial was made at a load of 46,500 lbs. and the bars failed at a load of 47,000 lbs. The ninety-first trial was made at a load of 47,000 lbs. and the bars failed at a load of 47,500 lbs. The ninety-second trial was made at a load of 47,500 lbs. and the bars failed at a load of 48,000 lbs. The ninety-third trial was made at a load of 48,000 lbs. and the bars failed at a load of 48,500 lbs. The ninety-fourth trial was made at a load of 48,500 lbs. and the bars failed at a load of 49,000 lbs. The ninety-fifth trial was made at a load of 49,000 lbs. and the bars failed at a load of 49,500 lbs. The ninety-sixth trial was made at a load of 49,500 lbs. and the bars failed at a load of 50,000 lbs. The ninety-seventh trial was made at a load of 50,000 lbs. and the bars failed at a load of 50,500 lbs. The ninety-eighth trial was made at a load of 50,500 lbs. and the bars failed at a load of 51,000 lbs. The ninety-ninth trial was made at a load of 51,000 lbs. and the bars failed at a load of 51,500 lbs. The hundredth trial was made at a load of 51,500 lbs. and the bars failed at a load of 52,000 lbs.

RESULTS AND DISCUSSIONS

Tests in all cases were continued until failure of the specimen occurred, either by splitting of the concrete, by compression failure of the concrete, or by the bar pulling through.

All smooth bar specimens failed by pulling through, but all specimens with 5/16 and 13/32-in cover developed longitudinal cracks on a plane through the plane of the bars, extending upward from the compression face.

The typical splitting (100%) of the deformed bar specimens was primarily in a plane through the smooth sides of the bars. Secondary splitting was at right angles. In specimens with the thinner cover and spacing, splitting was in a plane through the plane of the three bars, position of the lugs notwithstanding. Compression failure occurred in 6 specimens with covers of 9/16-in and less, 24 diameter embedment. Failure was conical, extending from the outer edges of the compression face inward and upward at an angle of about 30 degrees with the bars.

Tests in all cases were continued until failure of the specimen occurred, either by splitting of the concrete, by compression failure of the concrete, or by the bar pulling through.

All smooth bar specimens failed by pulling through, but all specimens with $\frac{3}{16}$ and $\frac{1}{4}$ -in cover developed longitudinal cracks on a plane through the plane of the bars, extending upward from the compression face.

The typical splitting (100%) of the reinforced bar specimens was typically in a plane through the smooth sides of the bars. Secondary splitting was at right angles. In specimens with too thin cover and spacing, splitting was in a plane through the plane of the three bars, position of the bars not alternating.

Compression failure occurred in 3 specimens with covers of $\frac{3}{16}$ -in and 1-in, 12 diameter and 12-in. Reinforcement was circular, extending from the outer edges of the compression face inward and upward at an angle of about 30 degrees with the bars.

TABLE NO. 7.

RESULTS OF 3/4 INCH ROUND DEFORMED BARS

<u>SPECIMEN NUMBER</u>	<u>BATCH NO.</u>	<u>COMPRESS. STR. PSI</u>	<u>EMBED. IN.</u>	<u>SPAC. & COVER</u>	<u>FIRST SLIP</u>	<u>ULTI- MATE</u>	<u>AVE. BOND</u>
109	3	4010	9	5/16	5900	5900	93
110(a)*	3	4010	9	3/8	P.F.*	7000	110
111(b)	3	4010	9	1/2	P.F.*	17200	270
115	2	3840	9	15/32	None	13200	208
116	2	3840	9	9/16	18500	18500	291
117(d)	2	3840	9	3/4	None	27000	424
121	1	3940	9	5/8	17500	18000	283
122	1	3940	9	3/4	21400	24100	379
121A(c)	1A	5220	9	5/8	18900	21000	330
122A(d)	1A	5220	9	3/4	22000	22800	358
123A	1A	5220	9	1	38800	40000	629
127	3	4010	13½	5/16	P.F.	9900	104
129(b)	3	4010	13½	1/2	None	19000	200
133	2	3840	13½	15/32	19000	19800	208
134	2	3840	13½	9/16	16000	20000	210
135(d)	2	3840	13½	3/4	At. Ult. 25000		262
141	1	3940	13½	1	41000	41400	434
139A(c)	1A	5220	13½	5/8	21000	22000	232
140A	1A	5220	13½	3/4	26300	27100	285
141A	1A	5220	13½	1	53500	54200	569
145	3	4010	18	5/16	Comp.*	11600	91
146(a)	3	4010	18	3/8	Comp.*	14500	114
147(b)	3	4010	18	1/2	None	22500	177

TABLE NO. 7.

RESULTS OF 1/4 INCH POINT MEASUREMENTS

STATION NUMBER	HAIR NO.	CORRECTION FT. PER IN.	SPAC. & CORRE.	ST. ST. ALL	HT. - WATER	AVG. GAGE
109	3	4.010	0	2.000	2400	43
110(a)*	3	4.010	0	2.1*	2500	110
111(b)	3	4.010	0	1.2*	17500	270
112	2	3.840	0	2.0/32	17500	208
113	2	3.840	0	2.1/32	18500	241
114(a)	2	3.840	0	2.1	17000	124
115	1	3.840	0	2.2	18000	203
116	1	3.840	0	2.2	21100	279
117(a)	1A	3.220	0	2.2	21000	330
117(b)	1A	3.220	0	3.1	22000	378
118	1A	3.220	0	1	24000	419
119	3	4.010	13 1/2	2.1/32	2500	104
120(b)	3	4.010	13 1/2	1.2	16000	200
121	3	3.840	13 1/2	1.2/32	18000	238
122	2	3.840	13 1/2	2.1/32	20000	270
123(a)	2	3.840	13 1/2	2.1/32	22000	266
124	1	3.840	13 1/2	1	21500	474
125(a)	1A	3.220	13 1/2	2	22000	472
126	1A	3.220	13 1/2	2.1/4	23000	523
127	1A	3.220	13 1/2	1	24000	569
128	1	4.010	18	2.1/32	24000	61
129(a)	3	4.010	18	2.2	24000	114
130(b)	3	4.010	18	1.2	23000	174

TABLE NO. 7.(cont'd)

RESULTS OF 3/4 INCH ROUND DEFORMED BARS

<u>SPECIMEN NUMBER</u>	<u>BATCH NO.</u>	<u>COMPRESS. STR.PSI</u>	<u>EMBED. IN.</u>	<u>SPAC.& COVER</u>	<u>FIRST SLIP</u>	<u>ULTI- MATE</u>	<u>AVE. BOND</u>
151	2	3840	18	15/32	18500	18650	147
152	2	3840	18	9/16	23800	24500	193
153(d)	2	3840	18	3/4	None	39700	312
157	1	3940	18	5/8	P.F.	34200	269
158(d)	1	3940	18	3/4	36400	36400	286
159	1	3940	18	1	45000	46900	369
157A(c)	1A	5220	18	5/8	28100	28600	225
158A	1A	5220	18	3/4	37500	38200	301
159A	1A	5220	18	1	58800	62600	494

*

- (a) - Averaged in plotting Figure 12- 3/8" cover.
- (b) - Averaged in plotting Figure 12- 1/2" cover.
- (c) - Averaged in plotting Figure 12- 5/8" cover.
- (d) - Averaged in plotting Figure 12- 3/4" cover.

*

P.F. - Slip just prior to failure.

*

Comp. - Compression failure at base of specimen.

Ultimate load and loads at first slip are in pounds;
average bond in pounds per square inch.

TABLE NO. 2. (Cont'd.)

RESULTS OF 3/4 INCH POWER SOFTENING TESTS

SPECIMEN NO.	DATE	CONCRETE STRENGTH (PSI)	STRENGTH (PSI)	STRENGTH (PSI)	STRENGTH (PSI)	STRENGTH (PSI)	STRENGTH (PSI)
151	2	3810	18	15/16	1800	1800	1800
152	3	3840	18	9/16	1800	1800	1800
153(a)	2	3810	18	3/4	1800	1800	1800
154	1	3840	18	3/8	1800	1800	1800
155(a)	1	3840	18	3/4	1800	1800	1800
156	1	3840	18	1	1800	1800	1800
157(a)	1A	3810	18	3/8	1800	1800	1800
158	1A	3810	18	3/8	1800	1800	1800
159	1A	3810	18	1	1800	1800	1800

(a) - Averaged in plotting figure 15-1/8 cover.
 (b) - Averaged in plotting figure 15-1/8 cover.
 (c) - Averaged in plotting figure 15-3/8 cover.
 (d) - Averaged in plotting figure 15-3/8 cover.

P.T. - Slip just prior to failure.

Temp. - Compression failure at base of specimen.

Ultimate load and loads at first slip are in pounds.

Average bond in pounds per square inch.

TABLE NO. 8.

RESULTS OF 1 INCH ROUND DEFORMED BARS

<u>SPECIMEN NUMBER</u>	<u>BATCH NO.</u>	<u>COMP. STR.PSI</u>	<u>EMBED. IN.</u>	<u>SPACE & COVER</u>	<u>FIRST SLIP</u>	<u>ULTI- MATE</u>	<u>AVE. BOND</u>
112	3	4010	12	5/16	None	13000	110
114(b)	3	4010	12	1/2	24000	24000	203
118	2	3840	12	15/32	P.F.	18600	157
119	2	3840	12	9/16	P.F.	26400	223
120(d)	2	3840	12	3/4	P.F.	35000	296
124A	1A	5220	12	5/8	27000	29150	246
125A(d)	1A	5220	12	3/4	37500	40200	340
126A	1A	5220	12	1	P.F.	51400	434
130	3	4010	18	5/16	Comp.	16400	92.5
131	3	4010	18	3/8	None	15200	85
132(b)	3	4010	18	1/2	None	26000	146
137	2	3840	18	9/16	None	29600	167
138(d)	2	3840	18	3/4	39000	40000	225
142	1	3940	18	5/8	None	37650	212
143	1	3940	18	3/4	P.F.	46400	262
144	1	3940	18	1	60000	62800	353
142A(c)	1A	5220	18	5/8	P.F.	36400	205
143A(d)	1A	5220	18	3/4	41000	42000	236
144A	1A	5220	18	1	None	57100	322
148	3	4010	24	5/16	Comp.	15000	63.5
149(a)	3	4010	24	3/8	Comp.	22600	95.5
150(b)	3	4010	24	1/2	None	29800	126
154	2	3840	24	15/32	Comp.	26900	114
155	2	3840	24	9/16	Comp.	32000	135
156	2	3840	24	3/4	P.F.	47500	200
160(c)	1	3940	24	5/8	None	38200	161

TABLE NO. 8. (cont'd)

RESULTS OF 1 INCH ROUND DEFORMED BARS

<u>SPECIMEN NUMBER</u>	<u>BATCH NO.</u>	<u>COMP. STR.PSI</u>	<u>EMBED. IN.</u>	<u>SPACE & COVER</u>	<u>FIRST SLIP</u>	<u>ULTI- MATE</u>	<u>AVE. BOND</u>
161(d)	1	3940	24	3/4	45000	47600	201
162	1	3940	24	1	None	80100	338
160A(c)	1A	5220	24	5/8	P.F.	48200	203
161A	1A	5220	24	3/4	None	49500	279
162A	1A	5220	24	1	None	84100	355

(Continued)

RESULTS OF 1 YEAR'S WORK

STATION	DATE	TIME	WIND	TEMP.	WIND	TEMP.	WIND	TEMP.
101	1910	10:00	10	10	10	10	10	10
102	1910	10:00	10	10	10	10	10	10
103	1910	10:00	10	10	10	10	10	10
104	1910	10:00	10	10	10	10	10	10
105	1910	10:00	10	10	10	10	10	10
106	1910	10:00	10	10	10	10	10	10
107	1910	10:00	10	10	10	10	10	10
108	1910	10:00	10	10	10	10	10	10
109	1910	10:00	10	10	10	10	10	10
110	1910	10:00	10	10	10	10	10	10
111	1910	10:00	10	10	10	10	10	10
112	1910	10:00	10	10	10	10	10	10
113	1910	10:00	10	10	10	10	10	10
114	1910	10:00	10	10	10	10	10	10
115	1910	10:00	10	10	10	10	10	10
116	1910	10:00	10	10	10	10	10	10
117	1910	10:00	10	10	10	10	10	10
118	1910	10:00	10	10	10	10	10	10
119	1910	10:00	10	10	10	10	10	10
120	1910	10:00	10	10	10	10	10	10
121	1910	10:00	10	10	10	10	10	10
122	1910	10:00	10	10	10	10	10	10
123	1910	10:00	10	10	10	10	10	10
124	1910	10:00	10	10	10	10	10	10
125	1910	10:00	10	10	10	10	10	10
126	1910	10:00	10	10	10	10	10	10
127	1910	10:00	10	10	10	10	10	10
128	1910	10:00	10	10	10	10	10	10
129	1910	10:00	10	10	10	10	10	10
130	1910	10:00	10	10	10	10	10	10
131	1910	10:00	10	10	10	10	10	10
132	1910	10:00	10	10	10	10	10	10
133	1910	10:00	10	10	10	10	10	10
134	1910	10:00	10	10	10	10	10	10
135	1910	10:00	10	10	10	10	10	10
136	1910	10:00	10	10	10	10	10	10
137	1910	10:00	10	10	10	10	10	10
138	1910	10:00	10	10	10	10	10	10
139	1910	10:00	10	10	10	10	10	10
140	1910	10:00	10	10	10	10	10	10
141	1910	10:00	10	10	10	10	10	10
142	1910	10:00	10	10	10	10	10	10
143	1910	10:00	10	10	10	10	10	10
144	1910	10:00	10	10	10	10	10	10
145	1910	10:00	10	10	10	10	10	10
146	1910	10:00	10	10	10	10	10	10
147	1910	10:00	10	10	10	10	10	10
148	1910	10:00	10	10	10	10	10	10
149	1910	10:00	10	10	10	10	10	10
150	1910	10:00	10	10	10	10	10	10

TABLE NO. 9.

RESULTS OF PLAIN BAR TESTS

<u>SPECIMEN & SIZE BAR</u>	<u>BATCH NO.</u>	<u>COMP. STR. PSI</u>	<u>EMB. IN.</u>	<u>SPACE COVER</u>	<u>FIRST SLIP</u>	<u>ULTI- MATE</u>	<u>AVE. BOND</u>
115S-3/4	2S	5005	9	15/32	12000	12100	190
116S-3/4	2S	5005	9	9/16	10900	11000	173
117S-3/4	2S	5005	9	3/4	7300	7300	116
133S-3/4	2S	5005	13½	15/32	15000	15000	157.5
134S-3/4	2S	5005	13½	9/16	18900	19800	208
135S-3/4	2S	5005	13½	3/4	21300	21600	217
151S-3/4	2S	5005	18	15/32	18100	18100	141.5
152S-3/4	2S	5005	18	9/16	23100	24800	195
153S-3/4	2S	5005	18	3/4	37500	38900	306*
119S-1	2S	5005	12	1/2	17200	17350	144
120S-1	2S	5005	12	11/16	16500	17000	141
136S-1	2S	5005	18	13/32	12000	12000	66.5
137S-1	2S	5005	18	1/2	21700	21800	121
138S-1	2S	5005	18	11/16	27500	29000	161
154S-1	2S	5005	24	13/32	17700	17700	73.6
155S-1	2S	5005	24	1/2	25000	25000	104
156S-1	2S	5005	24	11/16	36500	37600	156

* - Very rusty bars.

TABLE NO. 9.

RESULTS OF PLAIN BAR TESTS

BRIDGEMAN & SINE BAR	BATCH NO.	COMP. STR. PSI	WELD ST.	BRANCH GRADE	WIRE DIA.	WIRE WGT.	WELD WGT.
1152-3/4	25	5005	9	12/32	12/32	12/32	12/32
1162-3/4	26	5005	9	9/16	10/32	10/32	10/32
1172-3/4	27	5005	9	3/4	7/32	7/32	7/32
1332-3/4	28	5005	13/32	12/32	12/32	12/32	12/32
1342-3/4	29	5005	13/32	9/16	10/32	10/32	10/32
1352-3/4	30	5005	13/32	3/4	11/32	11/32	11/32
1512-3/4	31	5005	16	12/32	12/32	12/32	12/32
1522-3/4	32	5005	18	9/16	10/32	10/32	10/32
1532-3/4	33	5005	18	3/4	12/32	12/32	12/32
1102-1	34	5005	12	1/2	12/32	12/32	12/32
1202-1	35	5005	12	11/16	12/32	12/32	12/32
1362-1	36	5005	18	13/32	12/32	12/32	12/32
1372-1	37	5005	18	1/2	12/32	12/32	12/32
1382-1	38	5005	18	11/16	12/32	12/32	12/32
1542-1	39	5005	24	13/32	12/32	12/32	12/32
1552-1	40	5005	24	1/2	12/32	12/32	12/32
1562-1	41	5005	24	11/16	12/32	12/32	12/32

* - Very heavy bars.

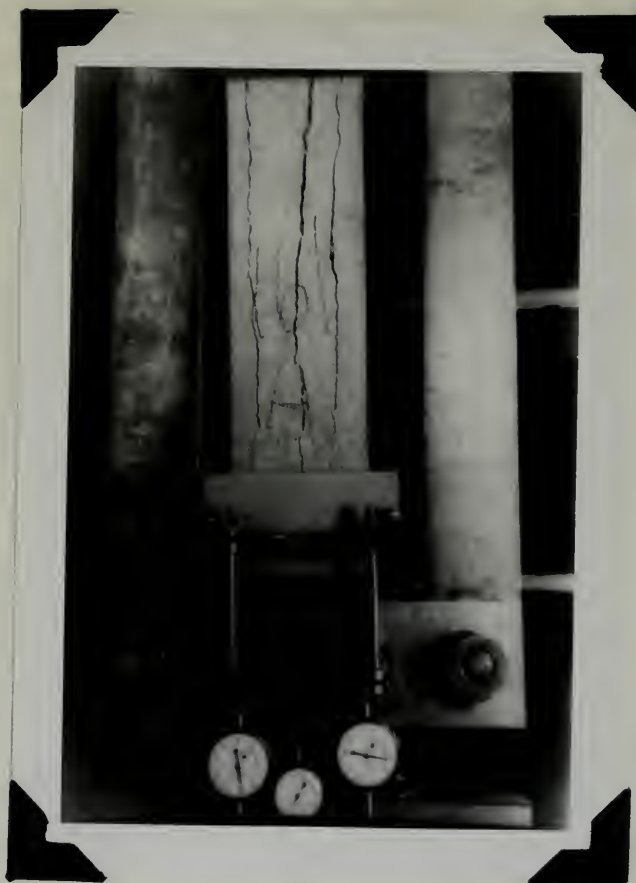


Figure No. 8.

Upper yoke with specimen showing a typical failure.
The specimen is a three-bar 1"Ø with 24" embedment and 3/4" spacing and cover.

212

Elmwood No. 8.

Upper part with typical bluish & typical leaves.

The specimen is a three-leaf 1 $\frac{1}{2}$ inch x 4 $\frac{1}{2}$ inch and 1 $\frac{1}{2}$ inch.

opening and cover.



Figure No. 8-A.

A typical failure. The specimens with 1" cover and 24" embedment failed with an explosive force but with the above general pattern. Failure generally was at right angles to the lugs as can be seen in the photograph.



Figure 8a. 3-4

A typical twill. The specimens with 16 cover
and 24° embossed failed with an oblique tear and with the
above general pattern. Twill generally was at right angles
to the tear as can be seen in the photograph.

Water gain was observed in all specimens, about 3% of the area under the bars not being effective in bond. Very little mill scale adhered to the concrete after failure.

The absence of a cap on the loaded end of the concrete resulted in no local failures.

Variation in maximum aggregate size caused no apparent variation in compressive strength or bond resistance.

Water gain was observed in all specimens, about 2% of the area under the bars not being effective in bond. Very little will scale adhere to the concrete after failure. The absence of a gap on the loaded end of the concrete resulted in no local failures. Variation in maximum aggregate size caused no apparent variation in compressive strength or bond resistance.

PLAIN BAR TESTS

The plain-bar tests were made as a reconnaissance to determine if plain bars might develop a higher bond efficiency or possibly higher bond values than deformed bars with corresponding values of thin cover. Unavoidable variations in the rust on the bars resulted in inconsistencies, but the over-all trend was the same as for deformed bars.

The splitting action was probably due to forces normal to the bars that were developed as slip progressed up the bar in the specimen. Some minute particles of the cement paste adhere to the steel in this slip. Their movement along the concrete surface resulted in a wedging action between concrete and steel, thus placing the surrounding concrete in tension and causing a splitting tendency in specimens of minimum cover which resulted in cracks on the narrow side.

The average bond value for all specimens of the same cover and spacing (each point representing the average of data for three specimens) is shown graphically in Figure No. 9.

Because of the few specimens tested, and the variations as mentioned above, reliable bonding efficiency curves could not be prepared.

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and steel, thus giving the protruding concrete in tension

and causing a splitting tendency in specimens of minimum cover

which resulted in cracks on the narrow side.

The average bond value for all specimens of the same

cover and spacing (each point representing the average of data

for three specimens) is shown graphically in Figure No. 7.

Because of the few specimens tested, and the variations

as mentioned above, reliable bonding efficiency curves could

not be prepared.

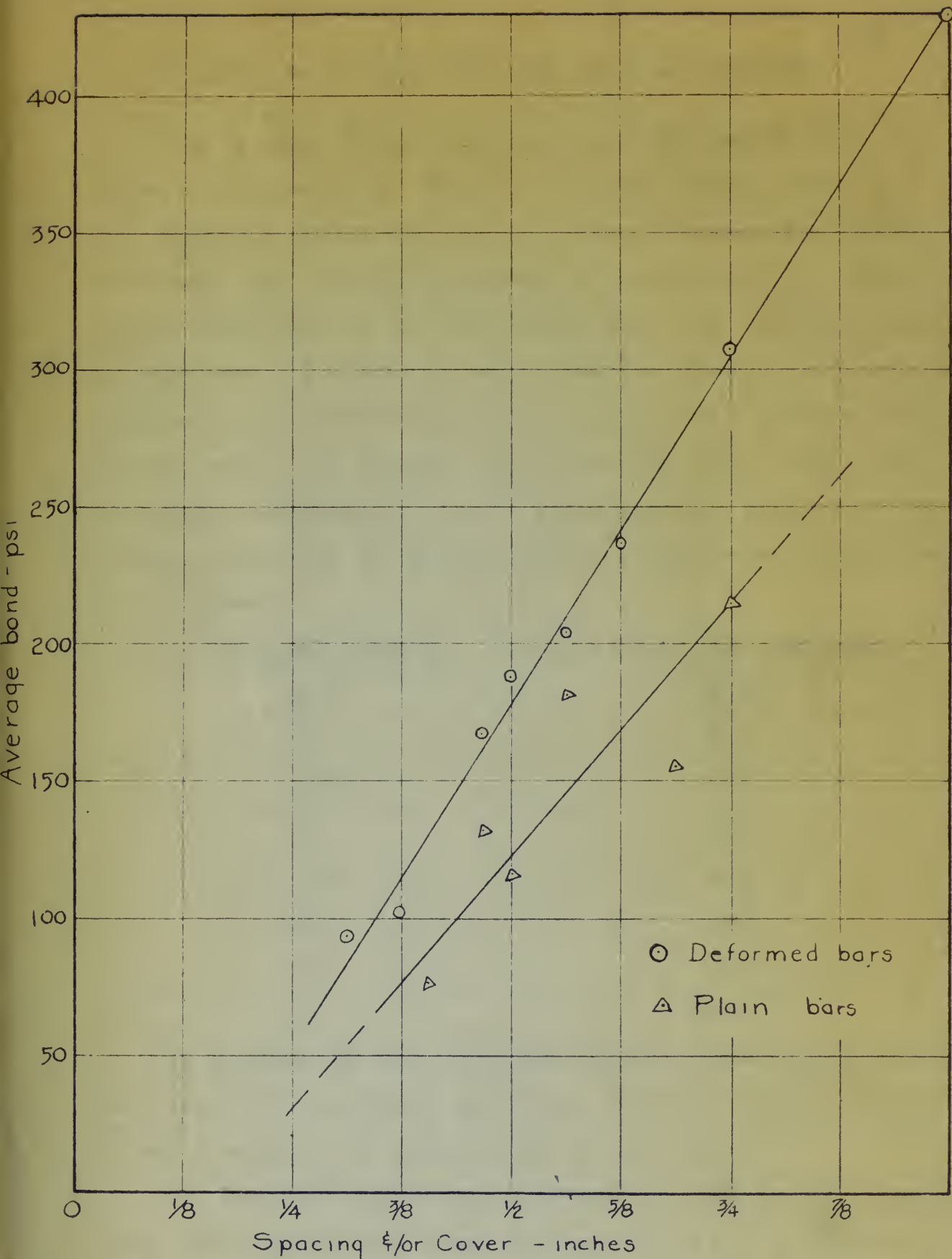


FIG. 9. AVERAGE BOND, VS SPACING AND/OR COVER

Points are average of average bond for all specimens of the same cover.

VARIATION OF AVERAGE BOND WITH COVER AND SPACING

The "Average Bond vs Spacing and Cover" relationship is shown in Figure No. 9. This is the grand average curve for all specimens tested wherein the points represent the average unit bond value for all specimens in the deformed bar series curve, regardless of the size of the bars used and the lengths of embedment. It is to be noted that the curve for the deformed bars gave very consistent points that plot in a straight line for the covers and spacing from 5/16-in to 1-in. Hence from a practical standpoint, it can be shown for this series that the bonding efficiency as based on 100% for 1-in cover and spacing is as follows:

<u>Cover and Spacing</u>	<u>% of Value for 1-in Cover</u>
5/16	18.6%
3/8	26.0
15/32	37.2
1/2	40.7
9/16	48.8
5/8	55.8
3/4	70.7
1	100.0

It is realized that for other factors constant, the curves will level off and become horizontal for some greater thickness of cover beyond that investigated in this series.

For the ACI Code design bond stress of 160 psi for the mix used, with unit safety factor and regardless of the stress in the steel, the minimum cover and spacing from the curve is 15/32-in for the deformed bars and 5/8-in for the plain bars investigated.

VARIATION OF AVERAGE BOND WITH COVER AND SPACING

The "Average Bond vs Spacing and Cover" relationship is shown in Figure No. 9. This is the same average curve for all specimens tested wherein the points represent the average unit bond value for all specimens in the deformed bar series curve, regardless of the size of the bars used and the length of embedment. It is to be noted that the curve for the deformed bars gave very consistent points that plot in a straight line for the covers and spacing from $\frac{5}{8}$ -in to 1-in. Hence from a practical standpoint, it can be shown for this series that the bonding efficiency as based on 100% for 1-in cover and spacing is as follows:

<u>Cover and Spacing</u>	<u>% of Value for 1-in Cover</u>
$\frac{5}{8}$	13.6
$\frac{3}{4}$	26.0
$1\frac{1}{2}$	37.2
$1\frac{3}{4}$	40.7
$2\frac{1}{2}$	48.8
$3\frac{1}{2}$	52.8
$4\frac{1}{2}$	70.7
1	100.0

It is realized that for other factors constant, the curves will level off and become horizontal for some greater thickness of cover beyond that investigated in this series.

For the ACI Code design bond stress of 180 psi for the mix

used, with unit safety factor and regardless of the stress in

the steel, the minimum cover and spacing from the curve is $1\frac{1}{2}$ -in

for the deformed bars and $\frac{5}{8}$ -in for the plain bars investigated

Stresses in all cases were in the elastic range for the steel used; hence, under similar conditions, the curve will give an indication of the unit bond to be expected when dealing with thin-shell precast concrete sections.

A breakdown of the "grand average" curve for the deformed bars is shown in Figure No. 10. Average bond vs cover and spacing is plotted for the three lengths of embedment for both $3/4$ -in and 1-in bars. The highest average bond for $3/4$ -in bars is for the 9-in embedment, followed by the $13\frac{1}{2}$ -in, and 18-in embedments. This increasing average bond with decreasing length of embedment is in agreement with the results of previous investigators. Curves for 1-in bars show similar results for lengths of embedment of 12-in, 18-in, and 24-in, respectively.

A comparison shows that for the 18" length of embedment with equal cover and spacing the $3/4$ " bars developed an average bond which varied from 10 to 15% greater than that for the 1" bars. Had the $3/4$ " and 1" bars developed equal average bond, the 1" bar specimens should have developed loads $33\frac{1}{3}\%$ higher than those with $3/4$ " bars due to ratio of areas per unit length. Failure, however, occurred by splitting, i.e., tension in the concrete, before equal bond was developed, but not at equal loads for the $3/4$ " and 1" bar specimens. If the splitting forces developed by the $3/4$ " and 1" bars had been the same for equal loads, the ultimate should have been the same for specimens with the same length of embedment, and the average bond for the $3/4$ " bars would have been $33\frac{1}{3}\%$ higher than that for the 1" bars. Actually the results are somewhere between the extremes of equal average

stresses in all cases were in the elastic range for the steel used; hence, under similar conditions, the curve will give an indication of the unit stress as indicated when dealing with thin-shell pressure vessels.

A breakdown of the "average" curve for the different

data is shown in Figure No. 10. Average bond vs cover and spacing is plotted for the three lengths of reinforcement for both 3/4-in and 1-in bars. The highest average bond for 3/4-in bars is for the 9-in embedment, followed by the 13 1/2-in, and 18-in

embedments. This increasing average bond with increasing length of reinforcement is in agreement with the results of previous

investigations. Curves for 1-in bars show similar results for lengths of embedment of 12-in, 18-in, and 24-in, respectively.

A comparison shows that for the 18-in length of embedment

with equal cover and spacing the 3/4-in bars developed an average bond which varied from 10 to 12% greater than that for the 1-

bar. And the 3/4-in and 1-in bars developed equal average bond, the 1-in specimens should have developed about 33-1/3% higher than

those with 3/4-in bars due to ratio of cross-sectional areas.

Failure, however, occurred by splitting, i.e., tension in the concrete, before equal bond was developed, but not at equal loads

for the 3/4-in and 1-in bar specimens. If the splitting forces

developed by the 3/4-in and 1-in bars had been the same for equal loads the ultimate should have been the same for specimens with the

same length of embedment, and the average bond for the 3/4-in bars would have been 33-1/3% higher than that for the 1-in bars. Actually

the results are somewhat between the extremes of equal average

bond and equal ultimate load for the specimens as shown by the "Ultimate Load vs Cover and Spacing" curves. (Figure No. 11).

The displacement of the curves indicates that, for the thin sections employed in this series, the bond is a function of the tensile strength of the concrete.

band and equal efforts for the expenditure of money by

the "Chinese Band" as cover and "spending" money. (Figure 2, p. 11.)

The displacement of the "Chinese Band" for the

this position requires is this matter, the band is a function

of the specific situation of the economy.

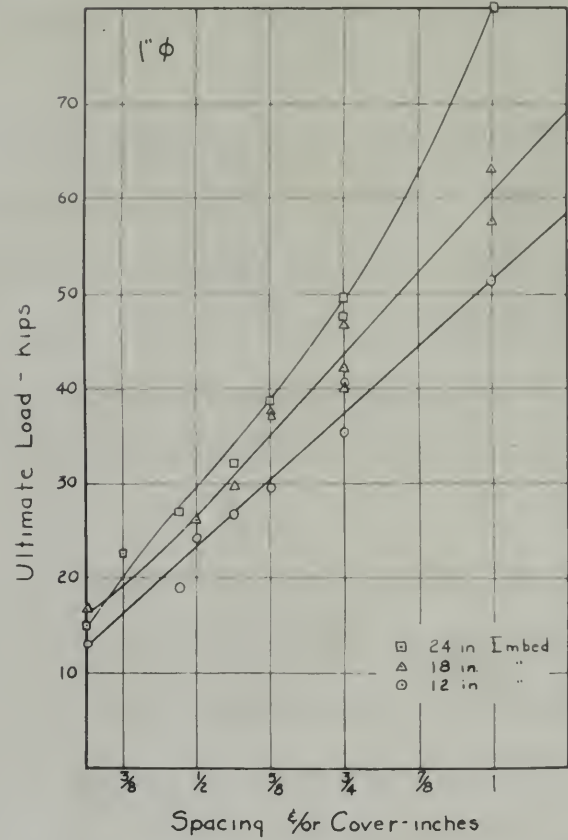
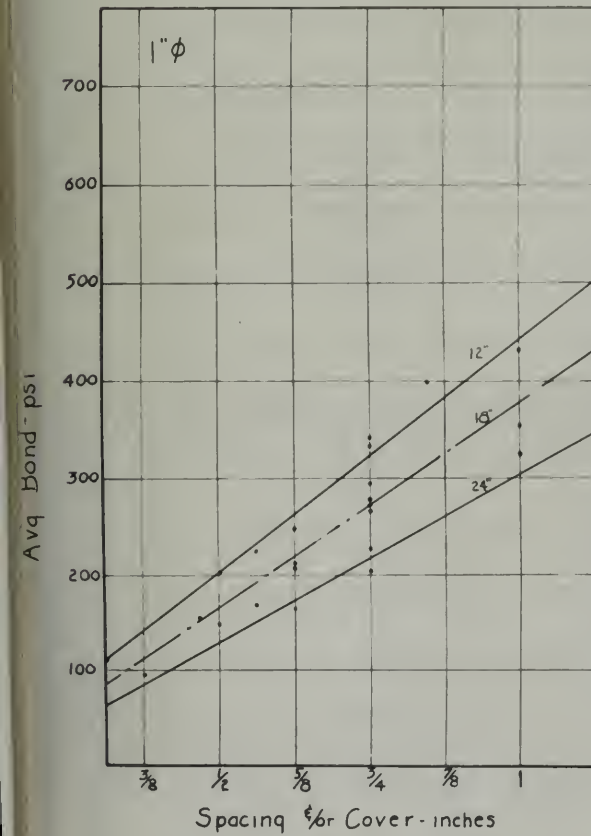
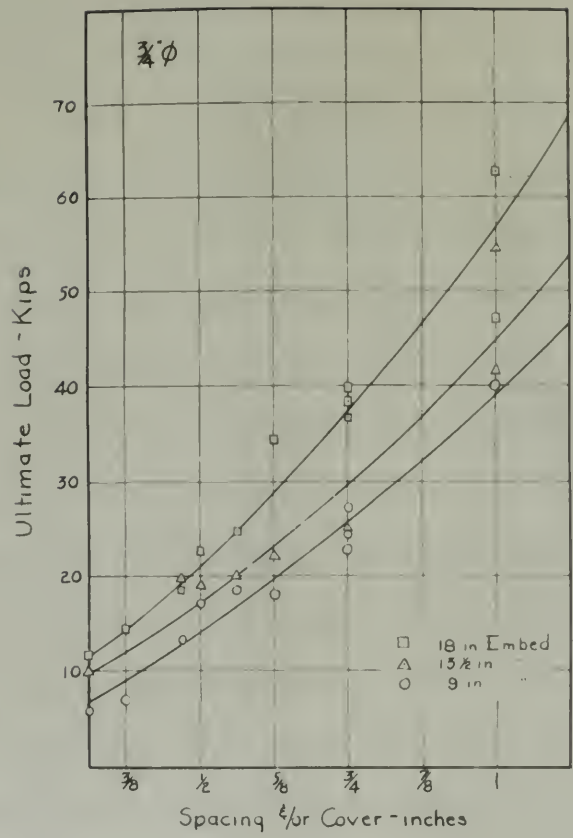
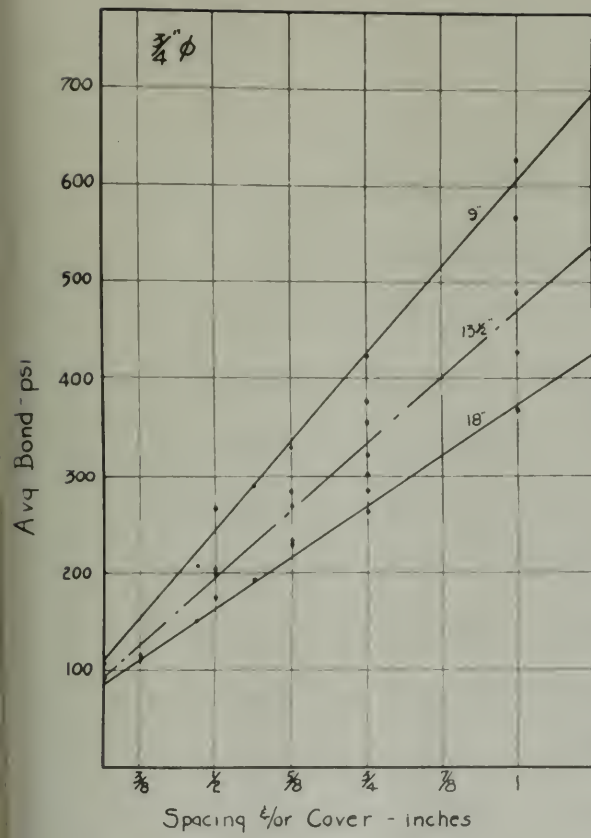


FIG. 10. VARIATION OF BOND WITH COVER

For deformed bars showing spread of points caused by variations in length of embedment

FIG. 11. VARIATION OF ULTIMATE LOAD

WITH COVER

For deformed bars with varying embedments

DISCUSSION ON CONCEPT OF BOND.

As pointed out by Gilkey, "bond stress can be present only in a region of changing stress in the steel or concrete. Bond is what makes stress transfer possible". The mechanical concept of bond is that of a combination of adhesive and frictional forces. As Kelley (14) states, "we must rely upon frictional resistance and not adhesion, for once a differential movement occurs between steel and concrete, adhesion is likely to be destroyed". Since stress transfer from the steel automatically insures a proportional amount of strain or relative movement between the concrete in compression and the steel in tension, the average bond as computed from ultimate load and total surface area of the reinforcing bar is a measure of frictional resistance. The adhesive bond having been successively released throughout the length of the bar prior to "first slip" or failure.

The frictional resistance is influenced by the roughness factor between the steel and concrete, that is by surface irregularities of the bar combined with any tightly adhering mill scale, rust, or cement paste. But, from the mechanics of the problem, this bonding force "must be proportional to the lateral pressures which exist or are developed between steel and concrete". (Kelley) (14).

As previously noted, 100% of the specimens using deformed bars were split by the wedging action of the lugs and the resulting tensile failure of the concrete. The questions of the mechanics of lug action are tantalizing. When does the lug actually become effective in increasing bond? What is the nature of this action? Obviously some slip between the bar and concrete must occur before

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The frictional resistance is influenced by the roughness factor between the steel and concrete, that is by surface irregularities of the bar combined with any closely adhering mill scale, rust, or cement paste. But, from the mechanics of the problem, this bonding force must be proportional to the lateral pressures which exist or are developed between steel and concrete". (Kelley) (1A).

As previously noted, 100% of the specimen being stressed bars were split by the wedging action of the jaws and the resulting tensile failure of the concrete. The questions of the mechanics of the action are tantalizing. When does the bar actually become effective in increasing bond? What is the nature of this action? Obviously some slip between the bar and concrete must occur before

the lugs become effective in bearing. Conversely, the effectiveness of the lug must depend to a certain extent upon the crushing or compressive strength of the concrete. That bond cannot be safely expressed as a linear function of concrete compressive strength has been repeatedly proved. Nevertheless, the fundamental consideration remains that stress must be transferred from the steel to the concrete. Be this by a locking action or by the pure concepts of friction as treated in mechanics, a certain amount of surrounding concrete mass is essential, in the former to withstand the shearing action resulting, and in the latter to maintain the required lateral force.

Kelley, (14) in his paper on factors influencing bond, states that bond strength is greatly influenced by volumetric changes within the paste of the concrete surrounding the bar. Gilkey, in his discussion of Kelley's paper, comments that "tests reported in Proceedings of Highway Research Board (1936) tend to confirm this conclusion relative to the greater bond resistance of concrete dry at test over that in a saturated condition. If this increase is valid and is due to shrinkage from drying the thicker the shrinking mass around the bar the greater should be the pressure exerted upon the bar".

This contraction due to drying after the initial shrinkage and set has occurred manifests itself in a gripping or "hugging" of the reinforcement as well as the aggregate in concrete. If the magnitude of the pressure exerted is proportional to the thickness of the surrounding concrete, obviously less bond is to be expected with thinner cover.

the idea become effective in being. Conversely, the effect-
iveness of the law must depend on a certain extent upon the
existence of compensating effects of the opposite. That some
extent be easily apparent as a linear relation of opposite
compensating effects has been repeatedly proved. Nevertheless,
the theoretical consideration remains that there must be
compensated from the world for the opposite. In case of a
looking action of by the same concept of friction is directed
in the opposite, a certain amount of unbalancing concept must be
essential. In the context of relation the existing action
resulting, and in the latter to maintain the negative balance
resulting.

Now Kelly, (14) in his paper on "Active Information Theory,"
states that bond strength is greatly influenced by volume
changes within the limits of the concepts surrounding the bond.
Likewise, in his discussion of Kelly's paper comments that "It is
reported in Proceedings of Highway Research Board (1954) that it
confirms this conclusion relative to the greater bond resistance
of concrete dry at least over that in a saturated condition. It
this increase is valid and is due to unbalance force drying . . .
the effect the saturated case around the bar the greater would
be the pressure exerted upon the bar."

This conclusion may be stated after the initial approach
and see the current manifest itself in a dynamic or changing
at the reinforcement as well as the applied in concrete. It
the magnitude of the pressure exerted is proportional to the
thickness of the surrounding concrete, obviously, less bond is to
be expected with thinner cover.

Regardless whether or not the above contentions are in themselves plausible, or if variations in lug sizes and shapes, in the homogeneity of the concrete paste, in the methods of placement and vibration, in the method and length of curing, or in the magnitude of the "released" adhesive force affect the bonding strength in a varying and somewhat unpredictable manner, the contention that increasing the cover will increase the bond up to a more or less definite limit, is, we believe, clearly shown in Figures Nos. 9 and 10.

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 themselves plausible, or if variations in the sizes and shapes,
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 or in the magnitude of the released stresses have affected
 the bonding strength in a varying and somewhat unpredictable
 manner, the contention that increasing the cover will increase
 the bonding to a more or less definite limit, is, as before,
 clearly shown in Figure Nos. 9 and 10.

STRESS-SLIP RELATIONS AS A MEASURE OF BOND EFFICIENCY

Believing that bar stress is a more reasonable criterion for bond efficiency in terms of slip at the loaded end than stresses developed by the bars at some arbitrarily selected value of slip, the method adopted by Clark (25) has been applied in this study.

The stress-slip relation, as measured at the loaded end (the slip of the three bars averaged and equal loads assumed in each of the three bars) is shown in Figure No. 12. Since plots for the various lengths of embedments showed no variation in slope as established by (Clark) (25), due probably to the extremely thin covers employed, curves represent average values for several specimens, lengths of embedment being neglected.

It is to be noted that the rates of stress transfer in these tests did not equal those obtained by Clark for values of slip up to $4/1000$ -in. For values of slip of $5/1000$ -in and more, however, specimens with covers of $3/4$ - and $5/8$ -in continued to load and the stresses exceeded the values of stress obtained with the top bar in Clark's series which had depth of concrete above the bar more nearly corresponding to that in this series. This is not true in specimens with covers of $1/2$ - and $3/8$ -in in which the maximum stress is below that he obtained. For comparison a graph is reproduced from Clark. (Figure No. 13).

The rates of stress transfer from steel to concrete are clearly shown in the figure and the flattening of the curves for less than $3/4$ -in cover clearly demonstrates the increasing

STRESS-STRAIN RELATIONS IN A SERIES OF SAND SPECIMENS

Believing that the stress is a more reasonable quantity for sand specimens in cases of slip of the loaded and shear stresses developed by the sand at some arbitrarily selected value of slip, the method adopted by Clark (19) has been applied in this study.

The stress-slip relation, as measured at the loaded end (the slip of the three bars averaged and equal loads applied in each of the three bars) is shown in Figure No. 12. Since plots for the various lengths of specimens showed no variation in slope as established by Clark (19), and, possibly to the extremely thin covers employed, stress-strain curves were plotted for several specimens, lengths of specimens being indicated.

It is to be noted that the rates of stress increase in these tests did not equal those obtained by Clark for values of slip up to $\frac{1}{100}$ -in. For values of slip of $\frac{1}{100}$ -in. and more, however, specimens with covers of $\frac{1}{4}$ - and $\frac{3}{8}$ -in. compared to load and the stresses showed the values of stress obtained with the cap bar in Clark's series which had depth of concrete cover the bar was nearly corresponding to that in this series. This is not true in specimens with covers of $\frac{1}{2}$ - and $\frac{3}{4}$ -in. in which the maximum stress is below that obtained. For comparison a graph is reproduced from Clark (Figure No. 11).

The rates of stress transfer from steel to concrete are clearly shown in the figure and the flattening of the curves for less than $\frac{1}{4}$ -in. cover clearly demonstrates the importance

inability of the concrete to develop bond with decreasing values of cover and spacing.

Although this study was made with only one type of deformed bar, it is reasonable to expect similar trends with other types, especially since the plain bar specimens showed similar trends, but with greater slips for the same stresses.

Accepting "stress transfer rate" as a criterion of bonding efficiency, the curves clearly show the increasing rates for corresponding increase in cover. At exactly what value of slip the effectiveness or usefulness of a structure becomes doubtful depends upon the particular structure under consideration and the value of the design stress in the reinforcing steel. For a structure using relatively low stresses in the reinforcing steel, such as one intended to confine liquids and avoid leakage, the cracks and deflections must be kept small. Both of these imposed conditions are a function of the slip which must, therefore, be also maintained a minimum. For such structures the value of cover to be used must be such, that for the design stress, the corresponding slip is small; i.e., a point on the steeper region of the curve. Conversely, in members for which such conditions do not prevail, a thinner cover may be employed for a given design stress, permitting greater slips to occur before developing the required stress.

It is to be noted that the ultimate value of stress possible increases with increasing cover as well as the rate of stress transfer. This ultimate value is also a function of the length of embedment, as might be expected; but the initial rate of transfer is independent of the length of embedment.

inability of the concrete to develop bond with reinforcing

bars of cover and spacing.

Although this study was made with only one type of reinforcement, it is reasonable to expect similar trends with other types, especially since the basic law of concrete behavior is the same, but with greater slope for the same stresses.

According to stress transfer tests as a criterion of bond

efficiency, the curves clearly show the increasing trend for

corresponding increase in cover. It is noted that the value of the

the effectiveness of reinforcement of a concrete beam depends

depends upon the particular structure under consideration and

the value of the design stress in the reinforcing steel. For a

structure using relatively low stresses in the reinforcing steel,

such as one intended to confine liquid and avoid leakage, the

cracks and deflections must be kept small. None of these factors

conditions are a function of the slip which must, therefore, be

also maintained a minimum. For such structures the value of cover

to be used must be such, that the design stress, the concrete

potential slip is small; i.e., a point on the stress-strain curve

curve. Conversely, in members for which only conditions of deflection

control, a minimum cover may be employed for a given design stress

providing greater slip is shown before developing the required

stress.

It is to be noted that the ultimate value of stress possible

increases with increasing cover as well as the rate of increase

thereafter. This ultimate value is also a function of the length of

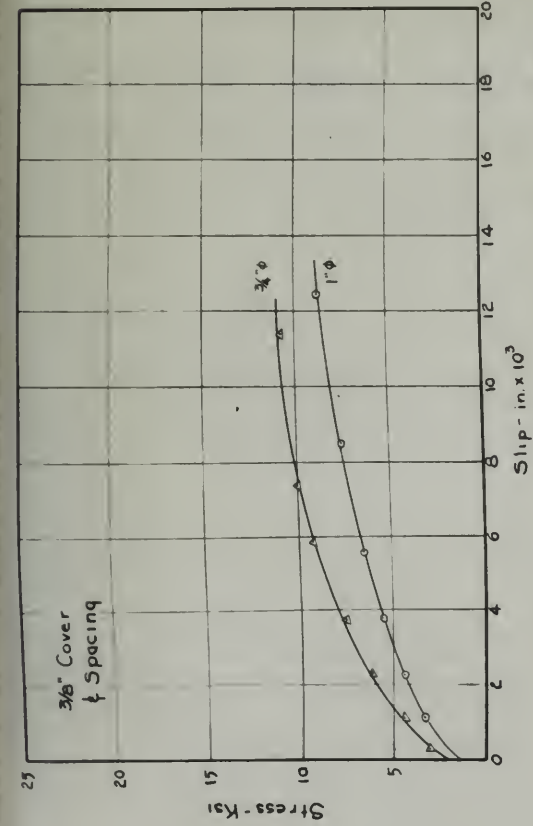
development, as might be expected; but the initial rate of increase

is independent of the length of development.

The bonding efficiency curves, (Figure No. 12), show a displacement of the curves for the 3/4-in bars above those for the 1-in bars. Therefore the curve shows that for the same stress in the steel there is greater slip in the 1-in bar than for the 3/4-in bar. For an equal surrounding mass of concrete the lateral forces exerted upon the bar to develop bond should be equal. The unit pressure or bond intensity, however, is less for the 1-in bar because of the greater surface area per unit length. Since bond is a result of stress transfer, for equal value of stress in the steel, greater slip must occur in the 1-in bars before this stress is developed by bond.

The bearing efficiency curves, (Figure 8-12), show a

displacement of the curves for the $\frac{3}{4}$ -in bars above those for the
1-in bars. Therefore the curve shows that for the same stress in
the steel there is greater slip in the 1-in bar than for the
 $\frac{3}{4}$ -in bar. For an equal surrounding mass of concrete the lateral
forces exerted upon the bar to develop bond would be equal.
The unit pressure on bond intensity, however, is less for the 1-in
bar because of the greater surface area per unit length, since
bond is a result of stress transfer. The equal value of stress in
the steel, greater slip would occur in the 1-in bars before this
stress is developed by bond.



Δ - 3/8 inch round bar.

\circ - 1" round bar

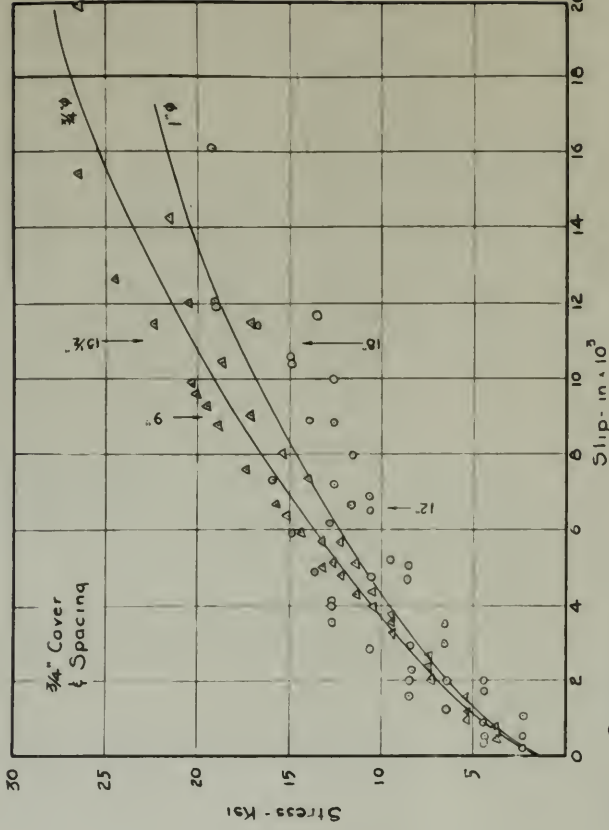
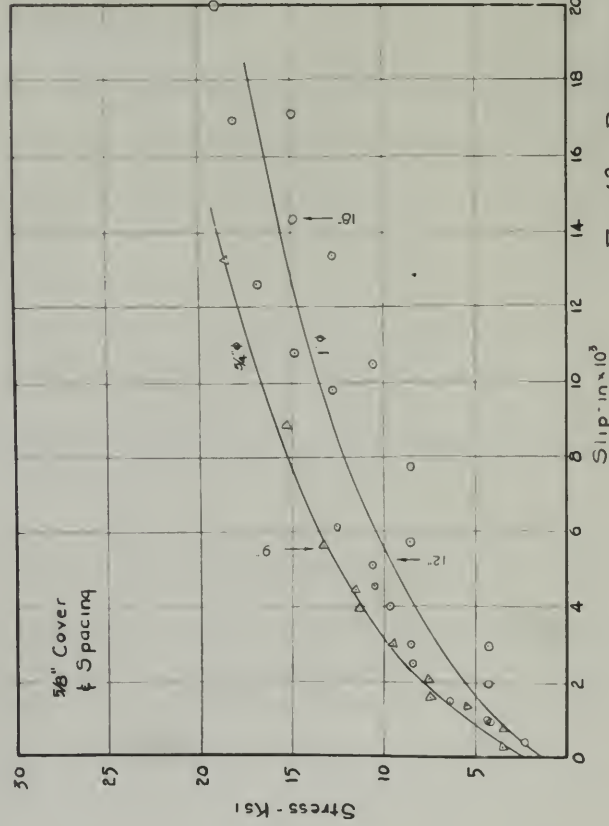
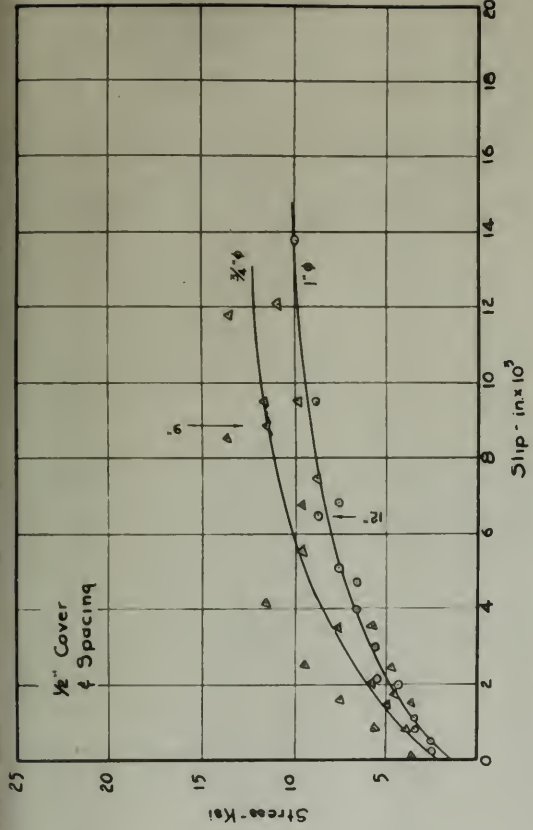


FIG. 12. BONDING EFFICIENCY CURVES

For numerous specimens of varying cover and spacing for 3/8 and 1 inch deformed bars with $\frac{1}{2}$ ratios of 12, 18, and 24. Increasing ultimate stress and increasing rate of stress transfer (Δ stress per unit slip) indicate increasing bonding efficiency with increasing cover. Slip measured at loaded end and plotted as rational average of slips for three bars per specimen. Stress = avg. stress in steel bar. Ultimate stresses developed are indicated for different lengths of embedment.

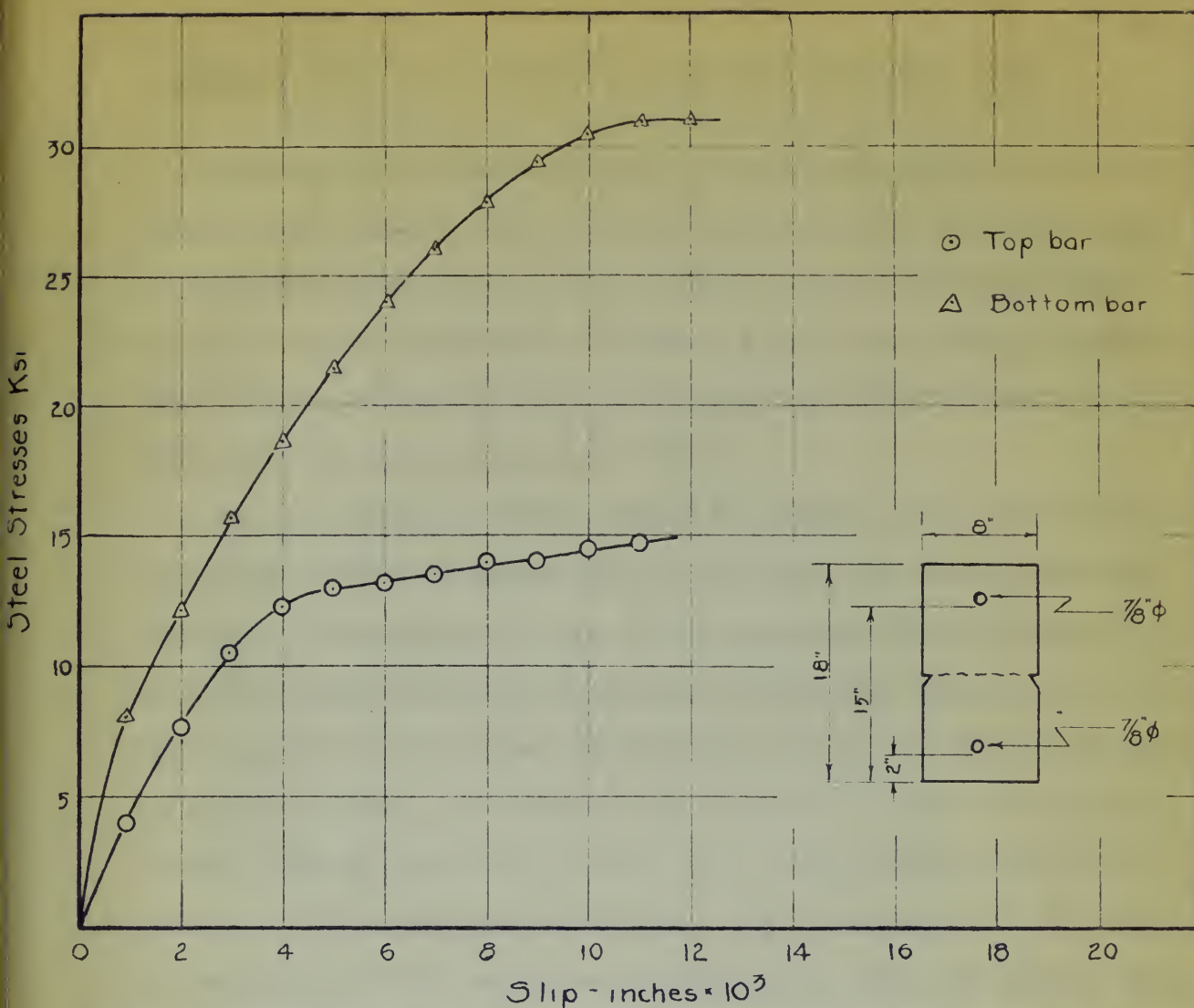


FIG. 13. BOND EFFICIENCY OF TYPE 16, $\frac{7}{8}$ " NOMINAL BAR. Embedded 16" in specimens cast as indicated. From Clark (ACI Journal, Feb. 47), presented for comparison.

The curves shown are for the same type of deformed bar used in these tests, each curve representing the average data obtained from three specimens.

In making comparisons with data gathered in this series due cognizance must be given the differences in bar diameter, length of embedment, and cover. Concrete averaged 5,600 psi at 28 days.

AVERAGE BOND AS A FUNCTION OF LENGTH-DIAMETER RATIO

Average bond values, based on ultimate load, are shown in Figures Nos. 14 and 15. as a function of L/D ratio. (Length of embedment/Bar dia.). The curves are plotted from data obtained on all specimens tested. Since time did not permit duplication of two batches of specimens, some curves are plotted from data on three specimens only.

In all cases, for the same L/D ratios, the 3/4-in bars developed higher average bond values for the same cover and spacing. Since the failure of the specimens was by tension in the concrete (longitudinal splitting) due to lug action and other wedging forces, it seems reasonable to explain this phenomena, at least in part, by considering tension in the concrete in planes through the axes of the bars. For equal covers and spacing, and length of embedment, the resistance of the concrete to tensile failure would be the same for 1-in and 3/4-in bars. The 1-in bar, because of its greater circumference and surface area per unit length, should develop the same splitting forces at lower unit stresses than will the 3/4-in bar.

Bond action, as previously discussed, is a transfer of stress along the bar and therefore of a progressive nature. That is, "there must be a progressive slippage or successive letting-go along the bar before either load or slip is transmitted to the unloaded end....". "Because of the inequality of bond stress along the bar, the length of embedment must be recognized as one of the significant variables when bond tests are compared." (Gilkey). (18).

AVERAGE BOND AS A FUNCTION OF LENGTH-DIAMETER RATIO

Average bond values, based on ultimate load, are shown in Figures Nos. 1A and 1B as a function of L/D ratio. (Length of embedment/Bar dia.). The curves are plotted from data obtained on all specimens tested. Since the 1-in and 1/4-in diam specimens of two batches of specimens, some curves are plotted from data on three specimens only.

In all cases, for the same L/D ratios, the 1/4-in bars developed higher average bond values for the same cover and spacing. Since the failure of the specimens was by tension in the concrete (longitudinal splitting) due to the action of wedging forces, it seems reasonable to explain this phenomenon at least in part, by considering tension in the concrete in planes through the axis of the bars. For equal covers and spacing, and length of embedment, the resistance of the concrete to tensile failure would be the same for 1-in and 1/4-in bars. The 1-in bar, because of its greater circumference and surface area per unit length, should develop the same wedging forces as lower unit stresses than will the 1/4-in bar.

Bond action, as previously discussed, is a function of stress along the bar and therefore of a progressive nature. That is, "there must be a progressive slip or relative sliding" along the bar before either load or slip is transmitted to the embedded end.... "Because of the insensitivity of bond stress along the bar, the length of embedment must be recognized as one of the significant variables when bond tests are compared." (Timothy). (16)

As pointed out by Gilkey and others, average bond decreases as length of embedment increases, even though the ultimate load developed increases with increasing length of embedment. The curves of average bond vs L/D ratios show further that this condition exists for each of the varying covers used, provided this cover gives sufficient area of concrete to avoid a compression failure of the specimen.

This series of curves illustrates quite clearly, we believe, the increased value of average bond which may be realized by increasing the cover and clear bar spacing within the limits shown. Average bond is not the maximum bond which may be successively developed along the bar, but it is the principle basis of design.

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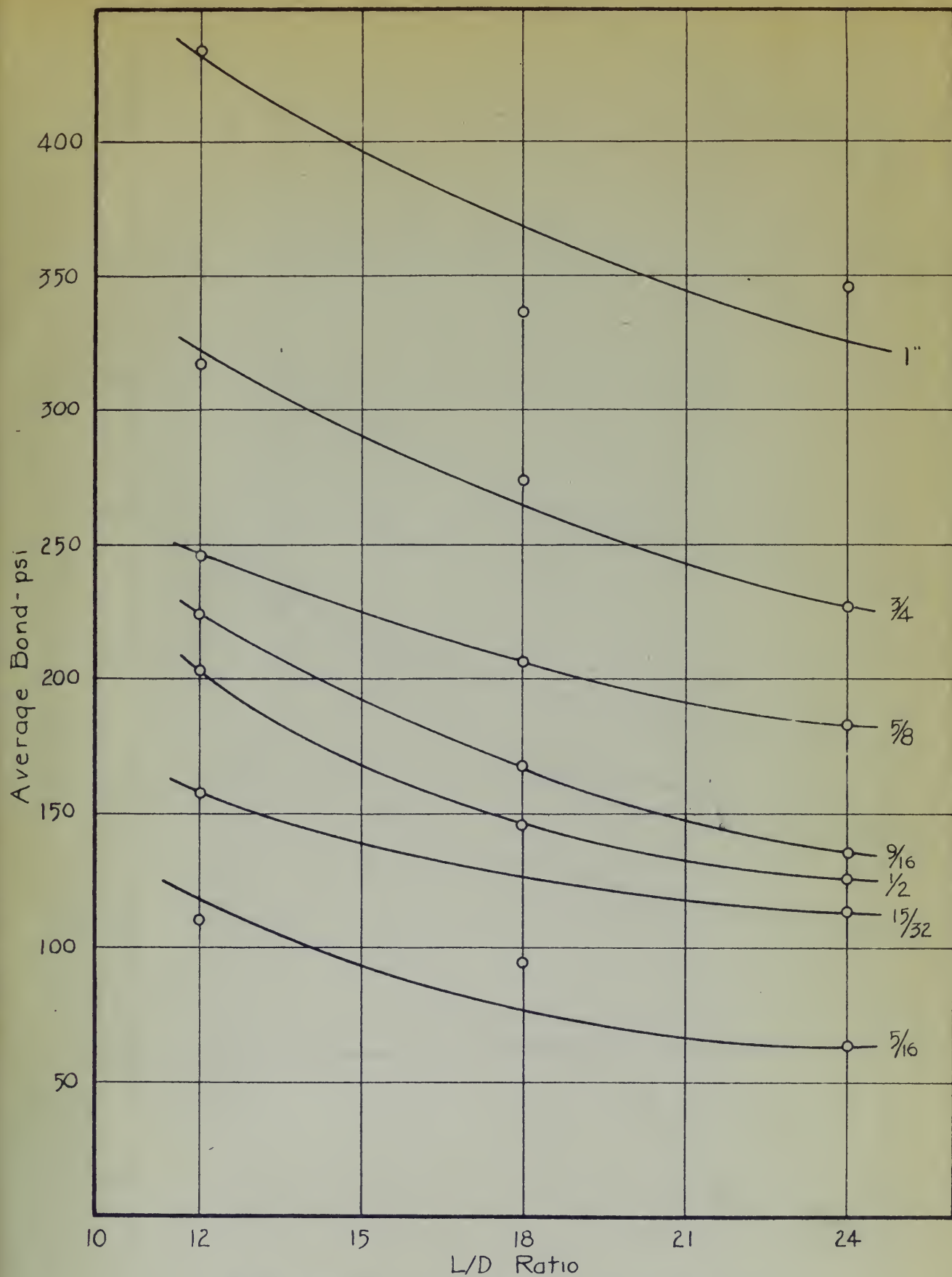


FIG. 14. AVERAGE BOND vs L/D ratio.

For varying cover and/or spacing for 1" round deformed bars

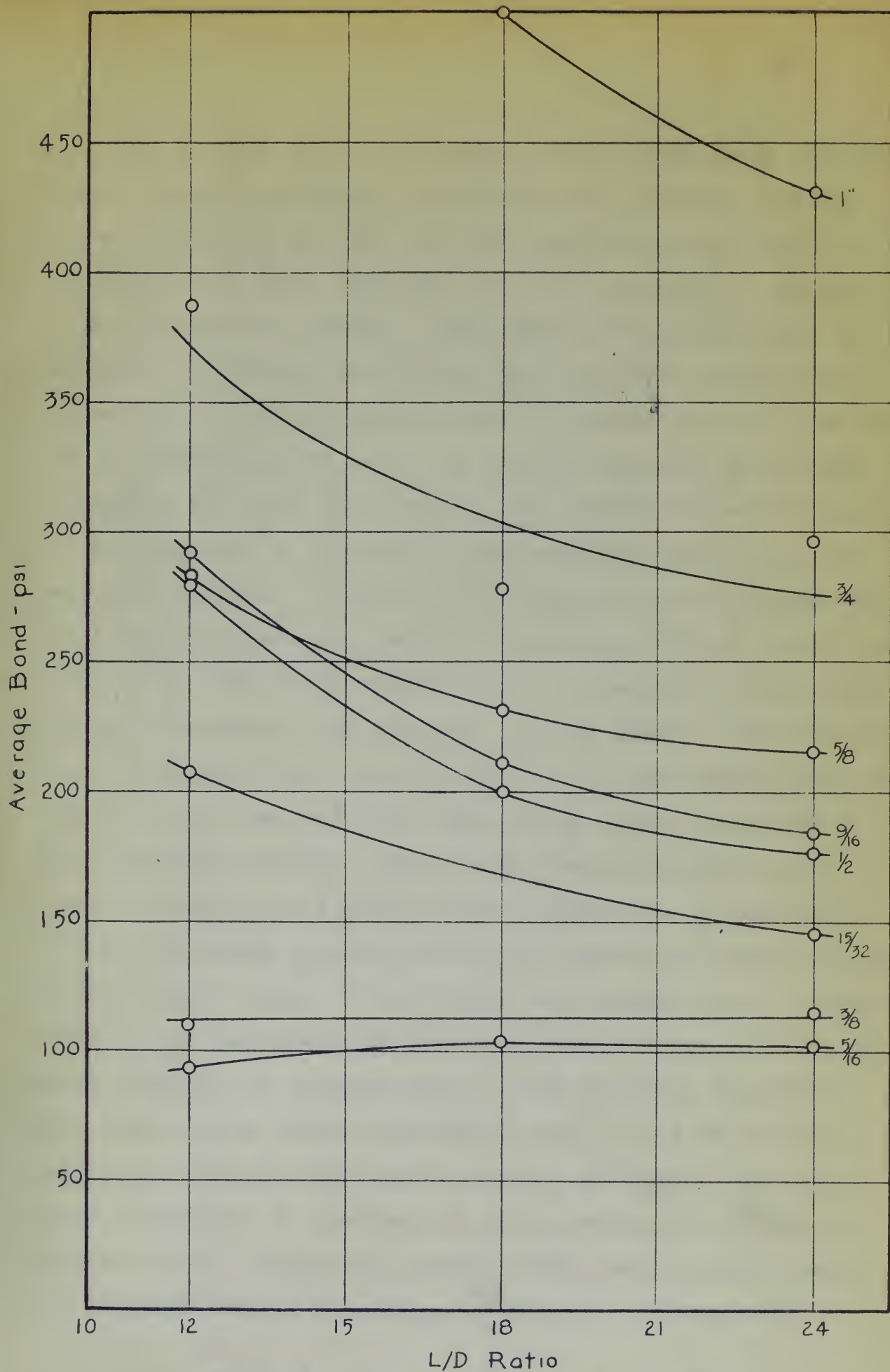


FIG. 15. AVERAGE BOND VS L/D RATIO
For varying cover and/or spacing for 3/4" round deformed bars

Time did not permit sufficient tests, namely those employing one-and two-bar specimens, to determine the relation existing between cover and spacing. In this report the values have, in all cases, been equal and hence the terms synonymous. In view of the observation, however, that failure of all specimens was attended by splitting between the bars, it seems reasonable to assume that a somewhat higher value of average bond could be realized by maintaining the cover, as employed in these tests, and increasing the clear bar spacing. The wedging forces developed by the lugs must be resisted by the concrete between the bars acting in tension. In these tests, since cover and spacing were equal, an outer bar had a width of concrete of " S "-in on the outside and " $S/2$ "-in on the inside, since the concrete in the clear spacing is effective for two bars. In the light of this explanation, it appears that the middle bar should have slipped more for a given stress than the outer bars, since it was surrounded by less effective concrete. This trend, though manifested to a limited degree was not sufficiently consistent to be conclusive.

The splitting of the specimens has heretofore been attributed to the wedging action of the lugs on the deformed bars. However, tests on the plain bars indicated the same tendency, but only for those specimens of minimum cover. It is possible, therefore, that cement paste tightly adhering to the plain bars produced sufficient roughness to cause a similar, but lesser splitting force. Splitting of specimens is not an unusual occurrence in pull-out tests. Gilkey (18) observed this action in his tests and remarked, "There is nothing to indicate, however, that

Time did not permit sufficient tests, namely those involving

one-end two-bar specimens, to determine the relation existing between cover and spacing. In this report the values have, in all cases, been equal and hence the terms synonymous. In view of the observation, however, that failure of all specimens was attended by splitting between the bars, it seems reasonable to assume that a somewhat higher value of average bond could be realized by maintaining the cover, as employed in these tests, and increasing the clear bar spacing. The wedging forces developed by the bars must be resisted by the concrete between the bars acting in tension. In these tests, since cover and spacing were equal, an outer bar had a width of concrete of $2\frac{1}{2}$ " in on the top edge and $2\frac{1}{2}$ " in on the inside, since the concrete in the middle spacing is effective for two bars. In the light of this explanation, it appears that the middle bar should have slipped more for a given stress than the outer bars, since it was surrounded by less effective concrete. This theory, though confined to a limited degree was not sufficiently consistent in its predictions. The splitting of the specimens has heretofore been attributed to the wedging action of the bars on the bottom bars. However, tests on the plain bars indicated the same tendency, but only for those specimens of minimum cover. It is possible, therefore, that cement paste tightly adhering to the plain bars produced sufficient roughness to cause a similar, but lesser splitting force. Splitting of specimens is not an unusual occurrence in pull-out tests. Gilkey (18) observed this action in his tests and remarked, "There is nothing to indicate, however, that

increased depth of cover can be justified as a device for preventing splitting from the wedging action of the lugs." This conclusion is predicated on the fact that failure of the specimen had already occurred, either by "first slip" on the free end of the specimen or by a slip of 1/100-in at the loaded end. Obviously, to develop a minimum bond resistance when failure is by splitting, sufficient cover must be used to obtain that value without failure.

For thin sections using plain bars, these tests indicate a minimum value of cover without splitting.

Considering length of embedment as shown in Figure 16 with average bond vs inches of embedment, curves for 1-in and 3/4-in bars form continuous curves.

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For this section using plain bars, these tests indicate a minimum value of cover without splitting. Considering length of embedment as shown in Figure 10 with average bond vs inches of embedment, curves for 1-in and 3/4-in bars form continuous curves.

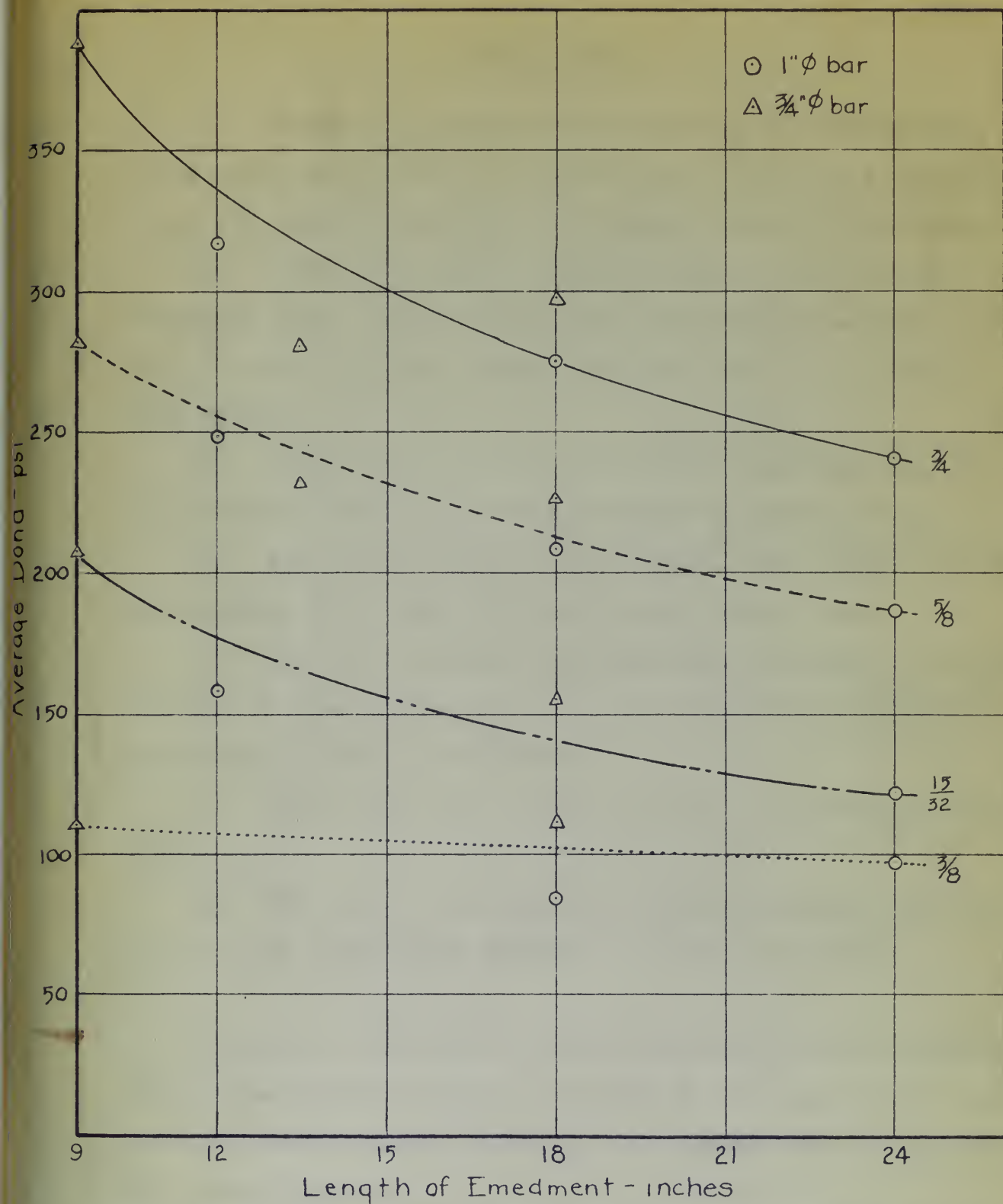


FIG. 16. AVERAGE BOND AS A FUNCTION OF EMBEDMENT

For the spacing and/or cover as shown. The dotted curve for $\frac{3}{8}$ " cover indicates minimum cover whereafter compression failures predominate.

CONCLUSIONS

1. Within the limits of these tests, bond resistance for a given size of bar is proportional to cover and spacing, i.e., the thicker the cover, the higher the bond resistance.
2. Within the limits of these tests, bond stresses developed with a given spacing and cover vary inversely as the size of bar, i.e., the greater the bar size, the less the bond stress.
3. Plain bars in thin-shell precast concrete exhibit bond characteristics and trends similar to deformed bars.
4. For a given stress in the steel, the thinner the cover and spacing, the greater the slip at the loaded end.
5. For the variations in compressive strength of concrete employed in these tests there was no appreciable variation in bond resistance or bonding efficiency.
6. Water gain, to a limited extent, is present in very stiff mixes.
7. The use of different size maximum aggregate had no influence on compressive strength or bond resistance.

Attention is called to the fact that the results obtained and the conclusions arrived at herein do not necessarily apply when the conditions are different from those that prevailed in this investigation.

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- Attention is called to the fact that the results obtained and the conclusions applied at herein do not necessarily apply when the conditions are different from those that prevailed in this investigation.

TOPICS FOR FURTHER INVESTIGATION

(All with respect to thin-shell precast concrete).

1. Completion of the test series to determine the relationship of one-, two-, and three-bar specimens.
2. Bond resistance developed at the end of 24 hours.
3. The possibility of increasing the bond efficiency by use of a steel wire spiral or steel mesh enclosing each reinforcing bar. The method should be applicable in the field to be of practical value.
4. Effects on bond of the use of sealing compounds to prevent the loss of moisture during curing.
5. Effects on bond of steam curing.
6. Bond efficiencies with other types of deformed bars.
7. Beam tests to check values of bond determined by pull-out tests.
8. The effects of using similar multi-bar specimens with cover equal to one-half the clear spacing.
9. Investigation of the minimum cover for different sizes of reinforcing bars.
10. The effect of different kinds of aggregates, crushed and smooth.

TOPICS FOR FURTHER INVESTIGATION

(All with respect to thin-shell process concrete).

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3. The possibility of increasing the bond efficiency by use of a steel wire spiral or steel mesh enclosing the reinforcing bar. The method should be applicable in the field to use of practical values.
4. Effects on bond of the use of sealing compounds to prevent the loss of moisture during curing.
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8. The effects of using similar multi-bar specimens with cover equal to one-half the clear spacing.
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A thorough study was made previous to any casting in order to gain sufficient knowledge of testing techniques and of the factors affecting bond so that the variables involved could be held as nearly constant as possible. Study was concentrated particularly in respect to the following items:

1. The concept of bond.
2. Nature of bar surface and surface coating.
3. Types of deformations on reinforcing steel.
4. Casting orientation with respect to lugs on the bars.
5. Rust on deformed bars.
6. Length of embedment.
7. Vibration and effects of over-vibration.
8. Mixing of concrete, aggregates, curing, and age.
9. Consistency of concrete.
10. Thin-shell precast concrete.
11. Testing procedures and techniques.

Following is a list, in chronological order, of references on bond and thin-shell sections:

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One of the earliest investigations covering bond which set the pattern for pull-out tests. For low strength concrete bond seems to be about proportional to the compressive strength of the concrete.

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Among the variables investigated are included age, mixture, consistency, and size of bar. Pull-out tests gave bond resistances $1\frac{1}{2}$ to 2 times that in beams. Withey ascribes the extra strength in pull-out specimens to compression in the concrete around the bar at the face of the block. Abrams' tests appear to discredit this contention.

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Among the variables investigated are included age, mixture, consistency, and size of bar. Pull-out tests have bond resistance 1 1/2 to 2 times that in beams. With any variation in the extra strength in pull-out specimens to compensation in the concrete around the bar at the face of the beam, Abrams' tests appear to disprove this contention.

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In addition Figure shows the pull-out type, modulus for
bond tests.

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water to travel upward through any porous material beneath
until the surface is reached. Moisture is trapped beneath the
horizontal surface, thus making the road between solid particles
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U.S. DEPARTMENT OF AGRICULTURE

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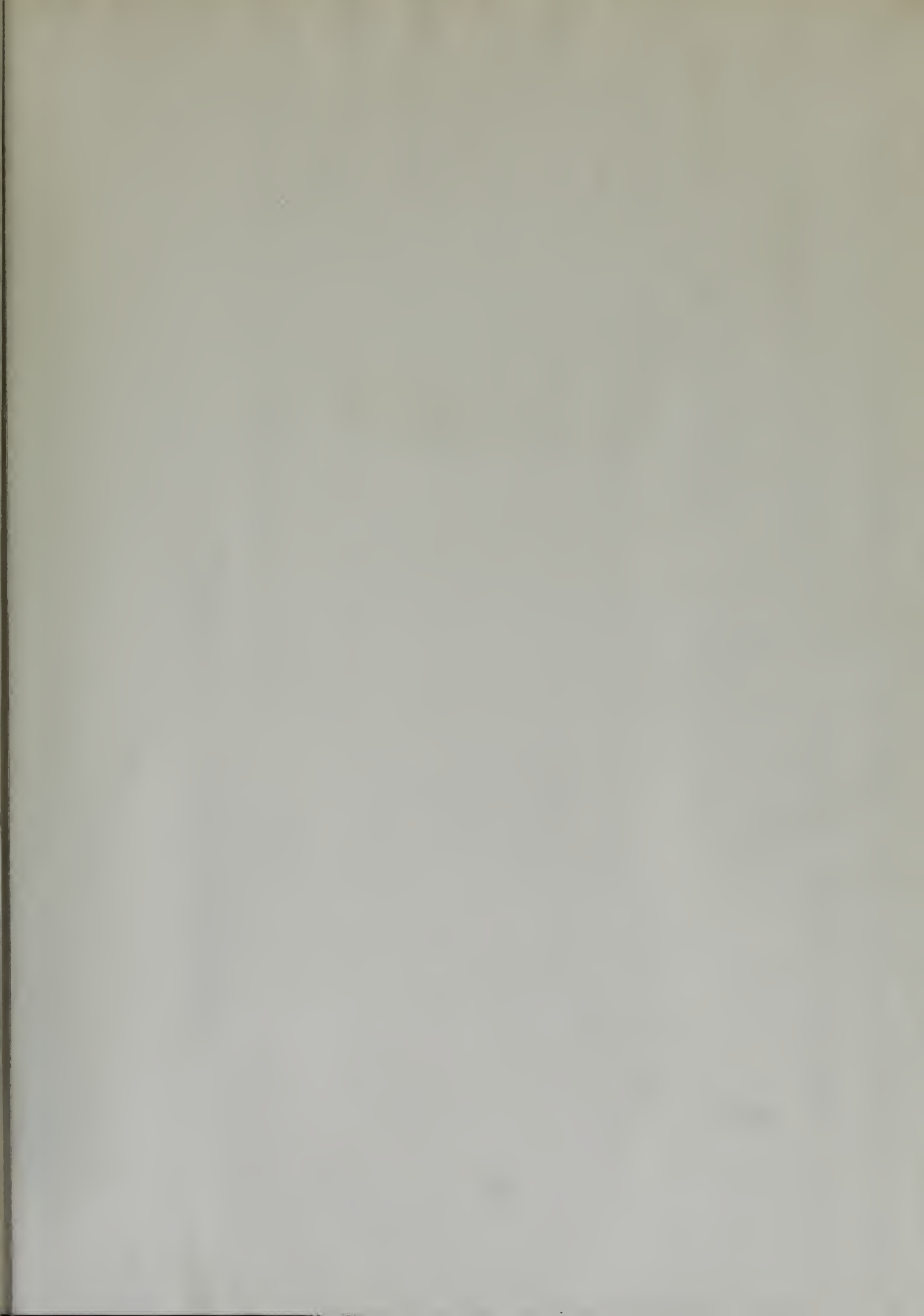
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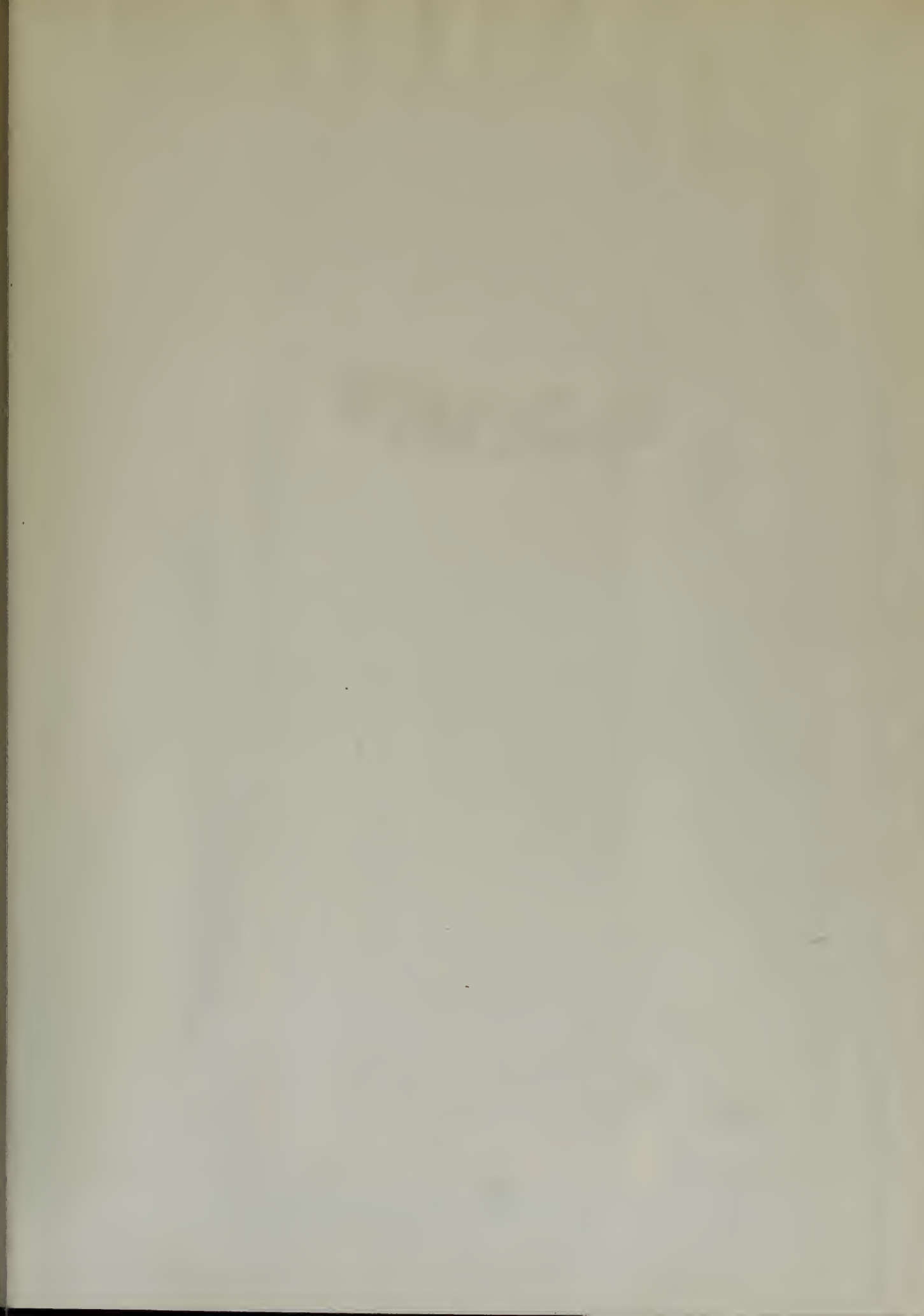
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